

REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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FLOOD FLOW CHARACTERISTICS

By C. S. JARVIS,* M. AM. SOC. C. E.

SYNOPSIS

The characteristics of flood flow that may have either general or special application and therefore must be considered in any preliminary studies for protecting a given locality are briefly stated, not by way of defining probable flood limits for individual cases, but of indicating methods of deduction that must be followed.

The available data that seemed reliable and valuable have been listed and platted to give a visual representation of the range of discharge per square mile for maximum recorded floods. Where possible, the portion of the maximum flow which represents the average annual maximum flood on each stream, has been indicated.

A method of attack is explained by which problems in several Central and Western Districts have been solved satisfactorily, both as to prevention of floods and protection of property.

In addition, there has been assembled in convenient form for reference a variety of representative basic and auxiliary data that are best portrayed diagrammatically.

The important public service which the Engineering Profession owes society in analyzing each local situation and in inaugurating or even restricting improvements as the future safety of the community may demand, is especially emphasized.

DESIGN OF STRUCTURES TO CROSS DRAINAGE SYSTEMS

An engineer endeavoring to design economical and safe bridges or similar structures is confronted with a maze of factors influencing the problem—scores of formulas and tables of data, hundreds of examples that serve as warnings, and thousands that appear satisfactory. Where records concerning the stream flow are lacking, he must rely on a study of the water-shed and channel, the alluvial bars and embankments, the drift lodged along canyon walls and in crevices, and the indefinite memories of observers. In general, he will be unable to reconcile the data thus gathered; and then remains the question of assigning proper weights to the various fragments of evidence.

Following such deductions, an important Western railroad company located nearly 100 miles of line along a valley floor, safely above the reach of the maximum assumed floods, only to experience its destruction within a few

NOTE.—Written discussion on this paper will be closed with the April, 1925, *Proceedings*. When finally closed, the paper, with discussion in full, will be published in *Transactions*.

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years. It was rebuilt on a higher line that would be safe against three or four times the maximum flow originally assumed. Similarly, various highway structures on which Federal aid was sought, had provided only one-half or one-fifth the proper capacity and during the torrential floods of 1923 some waterways that had been regarded as adequate proved able to carry only a small fraction of the maximum flow.

Proper design depends on knowledge of (1) the possible maximum flood; (2) the probable duration and frequency of its destructive stages; (3) the average yearly peak; and (4) the damage that will likely accrue from a channel overflow; as the fourth factor is a function of the first or the second, the importance of wisely forecasting the probable flood crests warrants patient labor and investigation.

CAUSES OF FLOODS

Floods result from (a) intensive rainfall; (b) prolonged storms that produce a combination of swollen tributaries; (c) rapid melting of snow such as accompanies warm rains; or (d) sudden release of impounded water. Even the last named may operate without human agency, as, for instance, when an ice dam breaks, or when a landslide forms a temporary impounding dam, to be cut through after the main stream has gathered greater volume and power. Too often floods are traceable to errors of judgment, or perhaps to the demand for economy, as in the cases of restricted spillways, reduced free-board for dams of the non-overflow type, and infringement on the river channel by permanent improvements.

DESTRUCTIVE STAGES

The highest flood stage of a stream ceases to be destructive when the endangered elements are either placed beyond its scope or protected from the violence of the current. The beneficent habits of the Nile would undoubtedly have been known as "destructive floods" if the ancients had established their homes and perishable property in the area subject to inundation, if they had arbitrarily encroached with levees on the required effective channel, or if occasionally the high flood stage had occurred at harvest time instead of during the planting season. Thus, the elements of unpreparedness and helplessness of victims overpowered by a relentless force, distinguish a calamity from an awe-inspiring display of latent power in proper control.

Within the past decade communities, after suffering property losses of several million dollars, have expended similarly large amounts for positive insurance against a recurrence. Just such calamities seem to be essential occasionally to crystallize public opinion in preventing channel encroachments that menace property and human lives. It is most regrettable that unscourged communities, exposed to the same peril, exhibit a hesitant attitude toward investigations based on new advances in the science of flood protection.

RAINFALL INTENSITY AND RESULTING RUN-OFF

If 1 in. of rainfall runs off uniformly in 1 hour, then each acre will yield 1 sec-ft., or each square mile will yield 640 sec-ft. If the shape and slopes

of the drainage basin, the lengths of the various tributaries, and the direction of movement of the storm unite to favor the worst combination, then the yield at some point might be a multiple of this rate. In most cases, due to absorption, percolation, and detention, the percentage that runs off is so low as to neutralize this cumulative effect of synchronous crests at junction points; yet there are apparently authentic records of yields from small areas mounting to 1 000 or even 4 000 sec.-ft. per sq. mile, continuing at such stages for only a few minutes, but meanwhile causing incredible damage.

In precipitous localities, especially those denuded of timber, undergrowth, and sod, the first storms that fall after a period of drought seem to yield a greater percentage of run-off than subsequent ones; a sudden penetration of moisture is resisted by the parched and dusty surface. Perhaps the outer inch or two of soil may be saturated and partly eroded, leaving the greater depths entirely dry. The undercutting of such banks along the gullies adds to the weight and momentum of the moving mass, and even retards percolation into the gravel and sand beds along the channel. The first small rivulet, preceding the main body like a pilot, may lubricate the trough so as to increase the natural velocity of the rising torrent, and may finally be overtaken by the flood wave. The great resistance of a rock-strewn channel retards the lower parts of the moving mass, and the upper volume then flows over and descends in front of the retarded portion in a rotary motion; thus, the crest, laden with logs and drift, is comparable to a huge Juggernaut as it crushes obstructions along its path. The content of sediment, fine unctuous clay, gravel, stones, and boulders, picked up from the stream bed according to their relative mobilities, serves to increase the specific gravity and destructive battering power of this first wave, and acts like a moving dam holding this shifting force in reserve, and quickly mounting to greater heights if the progress of the front line is checked momentarily. If the barriers hold, the lateral banks will be overtopped and the divided flood, disarmed of its heavy weapons, will seek out or carve for itself new channels. Examples of the effects of such mountain torrents are shown in Figs. 1 to 5. The value of such occurrences as warnings to communities similarly situated may even exceed the amount of damage caused if only steps are taken either to prevent a recurrence, or to control, guide, and dissipate the threatening crests.

DIVERSE FLOOD FLOWS

Between the combined avalanche and flood flow here described and the well ordered habits of other rivulets, especially those that have their sources largely in springs, or in snow deposits well protected against sudden thaws, there exists an endless variety of conditions. A north slope, a deep rocky gorge that entraps heavy drifts and provides security against direct sunlight, or topographic features such as mountain peaks on the windward side, serving alike to deflect the "chinooks" and the moisture-laden air currents, cooling them, and transforming the potential rain into snow, may modify run-off habits. As a result some main tributaries have flood problems of limited scope—rarely turbid and never departing far from mean flow—whereas others vary from a dry-channel condition to a peak exceeding that of the main river.

Such is the Gila River in Arizona; in its dry-channel stage it represents between 50 and 400 sec-ft. of underflow,* while as a maximum, its record has exceeded the Colorado River floods above the junction at Yuma, Ariz. The first severe storms in the summer of 1923 wrought havoc to railroad and highway traffic in various Western States. When a destructive flood crest tore out structures on up-stream tributaries of the Gila and approached the highway bridge at Florence, Ariz., a warning alarm was spread; but this flood never attained dangerous proportions nor even reached the first flood channel at this station, as the underground storage nearly absorbed it. A subsequent flood approaching with no higher crest was deemed of less importance and therefore was unheralded; yet it interrupted traffic at some of the lower crossings, because the underground storage space was already filled.

RAINFALL VARIANTS

Just as mountains lying across the path of moisture-laden atmospheric currents deflect them upward with a consequent chilling and condensation, so a sharp canyon or rift in a mountain range often causes an induced upward draft as through a chimney, elevating the warm moist air of the valleys to an altitude where the cold temperature results in copious rainfall.

If the convection currents moving upward have a velocity in excess of about 20 miles per hour, the condensation is blown back and scattered much like spray from a garden hose pointing into a violent wind; when the distribution is uneven some areas near the center of disturbance receive a double portion, while others are deprived of their normal supply. A notable example was the record-breaking rainfall in Central Texas, September 7 to 11, 1921, when 23.98 in. fell in 35 hours at Taylor, somewhat to leeward of the storm center, while Elgin, only 18 miles southward, and actually nearer the center of disturbance, recorded only 4 in. The area receiving more than 5 in. was nearly elliptical, the major axis extending along the pronounced Balcone fault zone for nearly 200 miles, from San Antonio to the Brazos River; Elgin in the midst of 15 and 20-in. rainfall areas had its portion, depleted to increase Taylor's abundance, doubtless by the action here described.†

RUN-OFF VARIANTS

The variations due to the presence or absence of vegetation; to the surface texture and condition; to the probability of sudden thaws, as from warm rains, or of intense precipitation on frozen ground; to the possibility of bank and flood-channel storage; to the direction of travel maintained by prevailing storms; to the altitude, temperature, annual rainfall, special topographic or meteorological features; and to the area, shape, and slopes of the drainage basin—all these variations must be considered in a given problem. The influence of the area appeals to the writer as the logical basis for a general formula, with the effect of the other variants grouped in the numerical coefficient corresponding to Kutter's or Chezy's *C*.

* "The Underground Waters of Gila Valley, Arizona", *Water Supply Paper No. 104*, U. S. Geological Survey, p. 49.

† "The Floods of Central Texas in September, 1921", *Water Supply Paper No. 488*, U. S. Geological Survey.

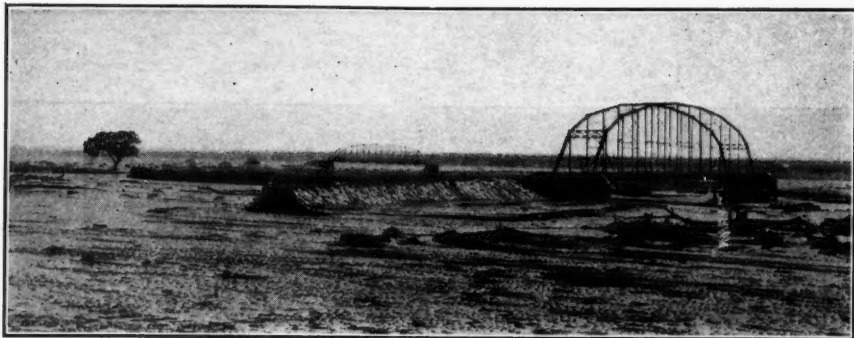


FIG. 1.—JUNCTION OF CANYON DIABLO WITH LITTLE COLORADO RIVER, ARIZONA, SHOWING EFFECTS OF THE SEPTEMBER, 1923, FLOOD.

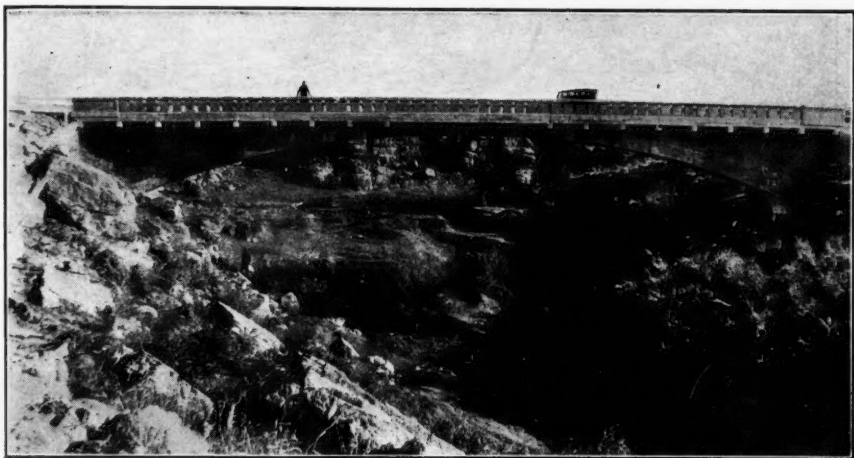


FIG. 2.—CANYON DIABLO, ARIZONA. FLOOD OF SEPTEMBER, 1923, BEACHED ABOUT TO THE CROWN OF THE BRIDGE.

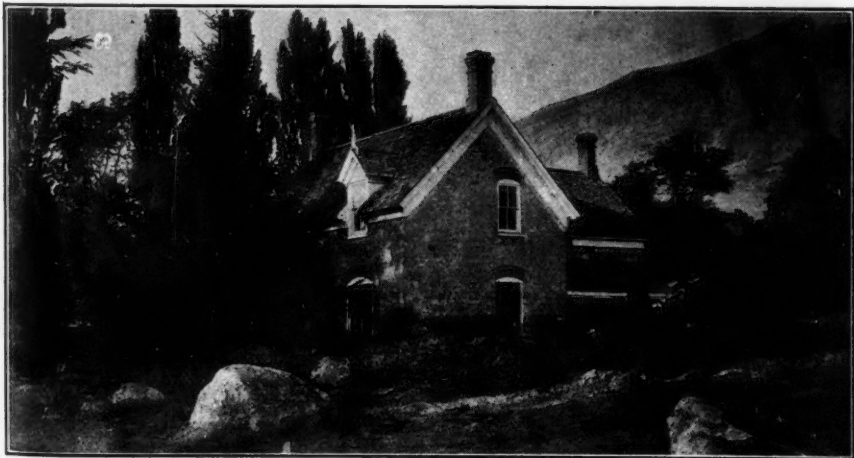


FIG. 3.—WILLARD, UTAH, FLOOD OF AUGUST, 1923. NOTE HEIGHT OF MUD DEPOSIT AGAINST HOUSE.



Fig. 1. View of the Great Salt Lake from the shore of the lake, looking southward.



Fig. 2. View of the Great Salt Lake from the shore of the lake, looking southward.



Fig. 3. View of the Great Salt Lake from the shore of the lake, looking southward.



FIG. 4.—FARMINGTON CANYON, UTAH. THE LARGE ROCK, 25 FT. IN DIAMETER, WAS MOVED 300 YD. BY FLOOD.



FIG. 5.—STATE HIGHWAY NEAR WILLARD, UTAH, AFTER THE FLOOD OF AUGUST, 1923.



FIGURE 1. A photograph of a light-colored, textured surface, possibly a piece of paper or a wall, showing a faint, irregular shape in the center.



FIGURE 2. A photograph of a light-colored, textured surface, possibly a piece of paper or a wall, showing a faint, irregular shape in the center.

DETERMINATION OF BASIC RUN-OFF FORMULAS

A careful study of diagrams on which are platted data for the yield, q , in second-feet per square mile of drainage area, as ordinates against the drainage areas, M , in square miles, as abscissas, indicates the relation:

$$q = \frac{C}{M^n} \dots \dots \dots (1)$$

in which C and n are constants.

If there is required, not the yield per square mile, but the total discharge from the basin, designated Q , it will be necessary to multiply each member of Equation (1) by M , that is,

$$q M = \frac{C M}{M^n}, \text{ or } Q = C M^{\frac{n-1}{n}} \dots \dots \dots (2)$$

which is one of the most convenient forms for practical field use.

Another easily applied formula results from dividing each member of Equation (2) by the assumed mean velocity to be attained at the section considered; the left-hand member will be the prescribed area of waterway, a , in square feet. Assuming a velocity of 10 ft. per sec., for example, and reducing C correspondingly to C' , there results:

$$\frac{Q}{10} = a = C' M^{\frac{n-1}{n}} \dots \dots \dots (3)$$

This fundamental form is followed, with slight variations, by many of the widely used and satisfactorily tested formulas for run-off or for area of waterway, with n taking values between 1.5 and 6 generally, giving rise to expressions, such as $\sqrt{M^5}$, and others much more complicated.

The bewildering number of factors, constants, and variants which have been so ingeniously devised to make the various formulas fit the needs of given conditions, apparently should be capable of reduction to a common denominator, so to speak, a means of translation without mastering each nomenclature or becoming an adept in the use of each of the various forms.

In devising a proper formula for run-off, the Myers type will be tentatively selected for modification or extension, and correlated with the recorded flood run-offs, as platted on Plate VIII. The Myers formula states $a = C \sqrt{A}$, in which A equals the area of the drainage basin, in acres, and C is a coefficient varying from 1 to 4. Transforming the area into square miles:

$$a = C \times 25.3 \sqrt{\frac{A}{640}} = C \times 25.3 \sqrt{M}$$

This agrees with Equation (3) if $C' = 25.3 C$ and $n = 2$.

Trying a value of C equal to 40 giving the product of $C \times 25.3$ as approximately 100, and assuming the velocity to be 10 ft. per sec., the resulting equation becomes:

$$Q = a V = 10\,000 \sqrt{M}$$

The resulting graph for Q (Plate VIII) is found to lie above all the platted points representing flood run-off rates except two, and to conform closely with the enveloping curve, thus justifying the assumption of C as 40 for a maximum.

Similarly, for the lower limit, the path of the point, $Q = 100 \sqrt{M}$, conforms well with the general slope, and lies above only 5% of the more than 1 000 platted flood run-off rates. This treatment suggests the idea that the modified Myers formula might be considered as $Q = R \sqrt{M}$, in which R is the expected rate of run-off, in second-feet, from 1 sq. mile, varying between 100 and 10 000.

To determine the required cross-sectional area of bridge opening, the allowable velocity would be used as a denominator, and,

$$a = \frac{R}{V} \sqrt{M} = S \sqrt{M}$$

in which, S is the coefficient for the sectional area of waterway.

Another method of comparing discharges would be according to the percentage, p , of the maximum for the corresponding area obtained from the modified Myers formula, $Q = 10\,000 \sqrt{M}$, which percentage is readily scaled for any point platted, as shown on Plate VIII; in that case the coefficient, R , in the previously given equation becomes $10\,000 p$; and $S = 1\,000 p$, if $V = 10$. Thus, the Colorado River below the Gila junction at Yuma would rate 5%; the Nile would be expressed as 4%; the Amazon, 50%; the Mississippi at Cairo, 21%; the Ohio at Paducah, Ky., 32%; the Gila both at Yuma and Florence, Ariz., 10%; the Salt River, prior to its control by storage reservoir, at the mouth and at Roosevelt, Ariz., 27%; Salado Creek, Texas, 117%, the highest rate platted; the Miami, Dayton, Ohio, 50%; Otay, Lower Otay Dam (causing failure), Calif., 40 per cent.

There may be other formulas just as capable of extension and modification to cover the entire field, but it will be difficult to find one more readily applicable, and agreeing with the platted run-offs more consistently than the modified Myers formula. In Fig. 6 is graphically illustrated the relationship between the best-known expressions for run-off.

In some cases a first approximation of run-off for a given district may be obtained by the use of data from Plate VIII, or from Table 2 (Appendix I), which is the same information in statistical form. As opportunity affords, further computations may be based on all the usual factors, representing expected rainfall intensities and duration, percolation, evaporation, detention, bank storage, slopes, forestation, tillage, elevation, shape of drainage basins, the time elements, geological formations, and other tangible influences.

VARIABLE FACTORS

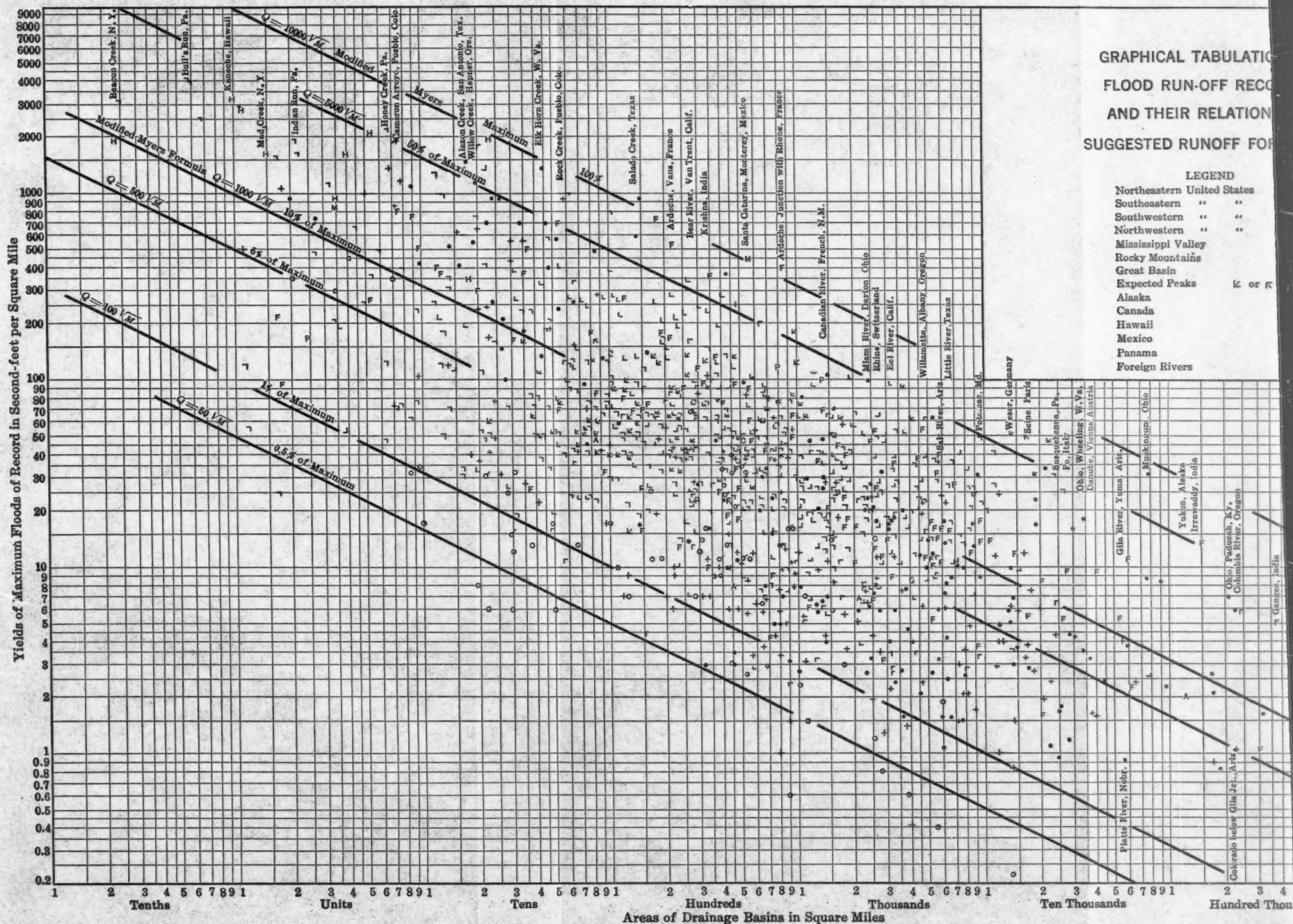
Among the considerations too often neglected, and so variable and elusive as to have escaped notice in standard formulas, are the following.

1.—*Inherent Similarity*.—Flood problems for small drainage areas may be essentially the same in various districts, even under widely dissimilar climatic conditions. If it be recognized that extreme rainfall and run-off rates for the arid and the humid regions of the United States, and, in fact, for nearly all

GRAPHICAL TABULATION FLOOD RUN-OFF RECORDS AND THEIR RELATION SUGGESTED RUNOFF FOR

LEGEND

- Northeastern United States
- Southeastern " "
- Southwestern " "
- Northwestern " "
- Mississippi Valley
- Rocky Mountains
- Great Basin
- Expected Peaks
- Alaska
- Canada
- Hawaii
- Mexico
- Panama
- Foreign Rivers



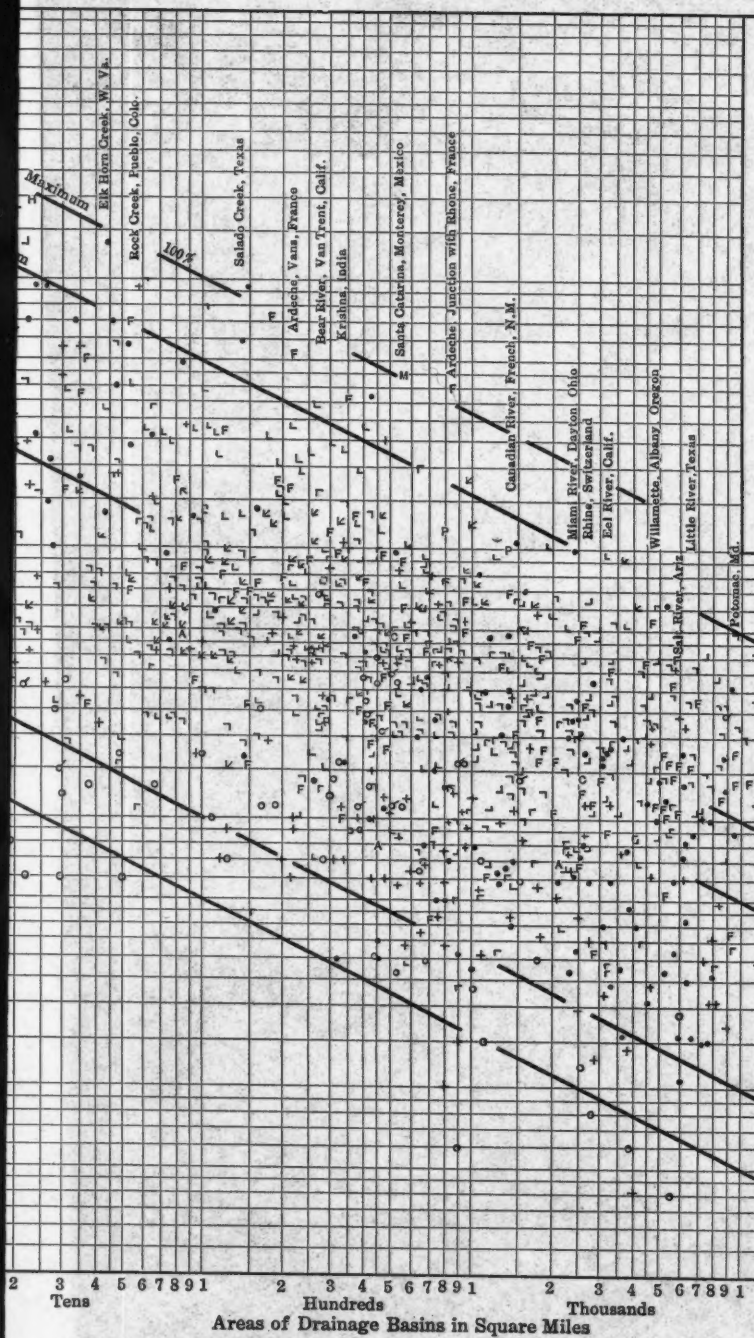
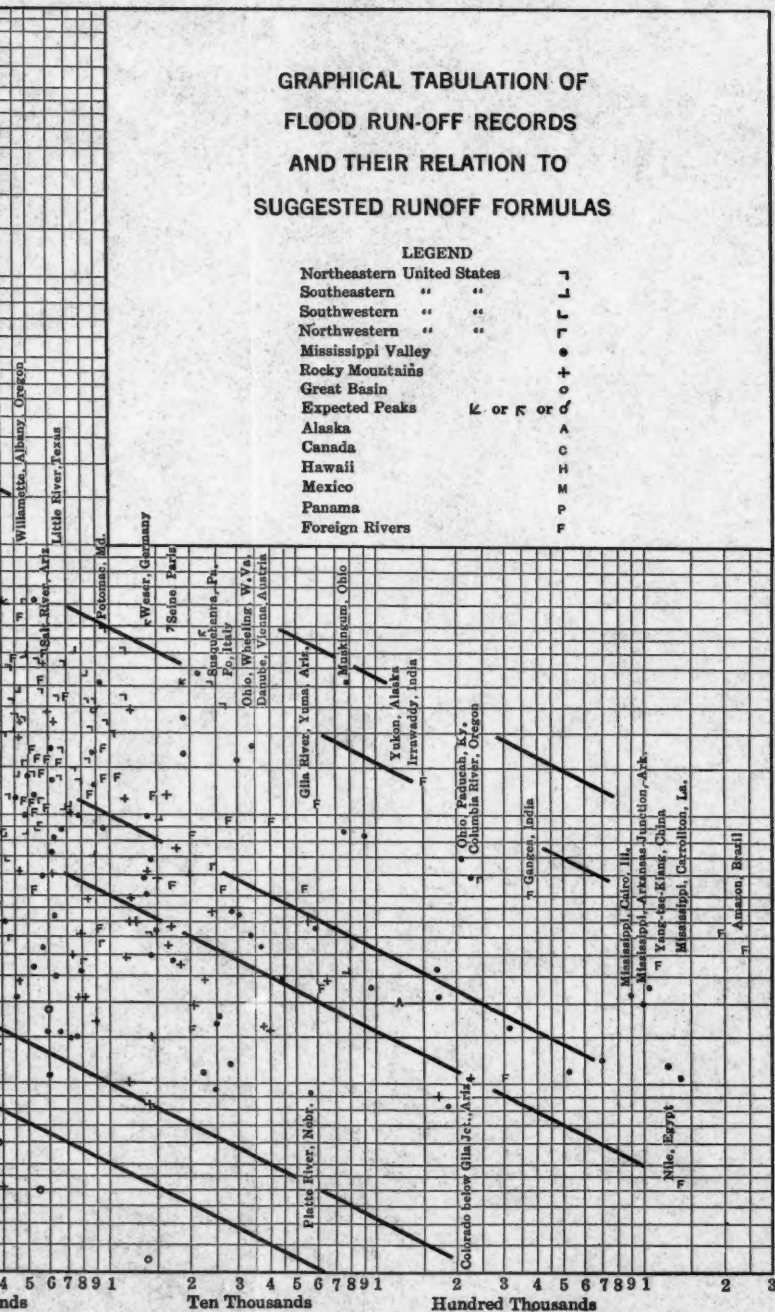


PLATE VIII.
PAPERS, AM. SOC. C. E.
DECEMBER, 1924.
JARVIS ON
FLOOD FLOW CHARACTERISTICS.

GRAPHICAL TABULATION OF
FLOOD RUN-OFF RECORDS
AND THEIR RELATION TO
SUGGESTED RUNOFF FORMULAS

LEGEND

Northeastern United States	7
Southeastern " "	1
Southwestern " "	1
Northwestern " "	1
Mississippi Valley	•
Rocky Mountains	+
Great Basin	o
Expected Peaks	κ or κ or σ
Alaska	A
Canada	C
Hawaii	H
Mexico	M
Panama	P
Foreign Rivers	F



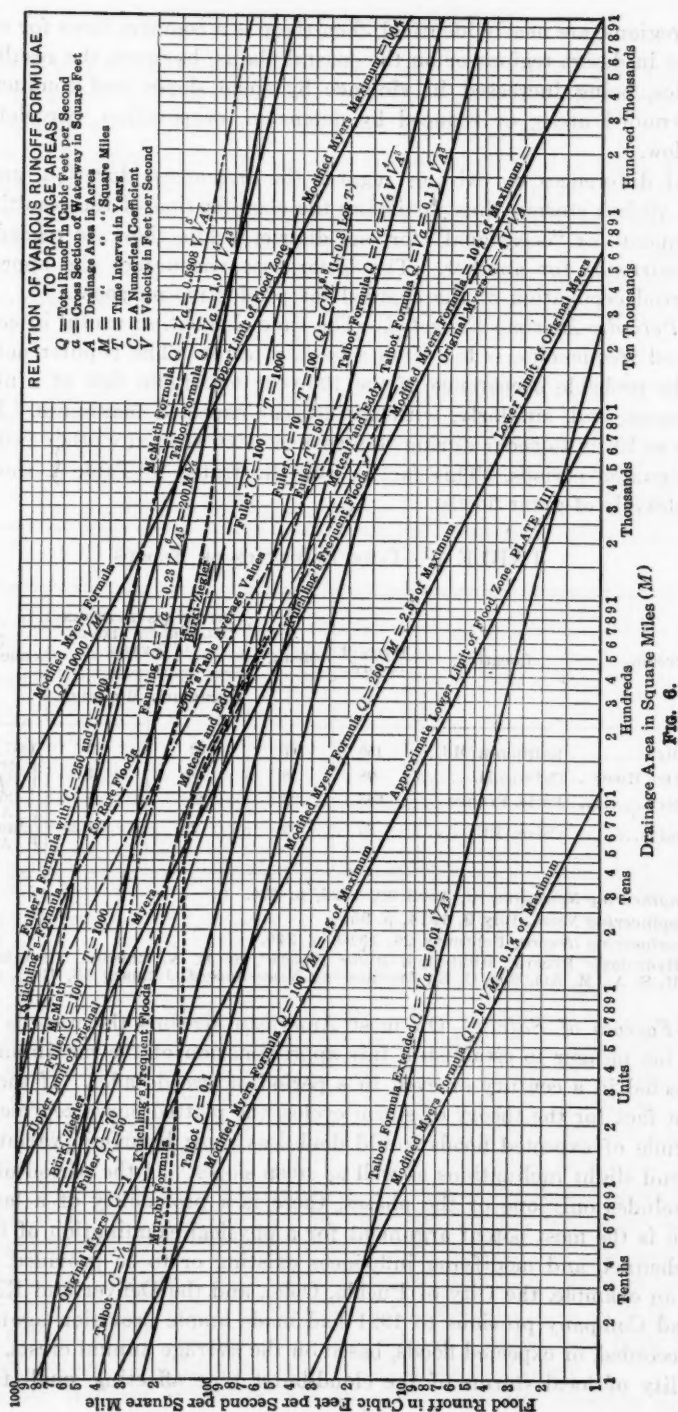


Fig. 8.

known regions, are nearly identical, then there is a common basis for estimates. Whether in desert regions or on the coastal plains, however, the resulting run-off varies, being increased in violence by steep slopes and confluent crests from denuded areas, or reduced by detention, evaporation, percolation, and under-flow.

Vital differences do exist as regards the frequency, duration, and extent of such violent storms, thus justifying the classification of the resulting floods as "frequent" or "occasional" for one district, while they are regarded as of rare occurrence for another. The latter case encourages the appropriation and partial occupation of the channel for productive purposes.

2.—*Periodic Maxima Expected.*—The time element, so useful in connection with flood frequencies, is too often a basis of error. The popular notion is to place the probable occurrence of the 100-year maximum flow at a number of years hence, and, similarly, with the 20-year expected maximum. Either of these is as likely to occur during this year or next as in any other two years of the designated periods. This fact is well illustrated in Table 1, showing the time intervals of great floods.

TABLE 1.—TIME INTERVALS OF FLOODS

Stream.	Location.	Years of record.	Mean.	INTERVALS BETWEEN DISASTROUS FLOODS, IN YEARS.		Notable concentrations.
				Maximum.	Minimum.	
Jones Falls*.....	Baltimore Md...	150	17	32	2	1837, '42, '58, '60
Los Angeles Rivert..	California.....	98	8	29	2	{ 1894, '86, '89; 1911, '14, '16
Seine River†.....	Paris, France...	265	29	82	3	{ 1879, '82, '85 April, 1912
Mississippi‡.....	Cairo, Ill.....	80	3	7	0.25	{ January, 1913 April, 1913

* *Engineering News-Record*, April 20, 1922, p. 654.

† *Engineering News*, July 9, 1914, p. 95.

‡ *Engineering Record*, February 26, 1910, p. 240.

§ "Hydrologic Record, Mississippi River Floods", by E. N. Chisolm, Captain, Corps of Engrs., U. S. A., M. Am. Soc. C. E., *Engineering News-Record*, January 18, 1923, p. 112.

3.—*Factors of Safety.*—On most American streams the records are altogether too meager to establish a fair approximation of the maximum flood to be expected in a century or even in a period of two decades. If there is any basis in fact for the theory of storm cycles, the platted curve representing the magnitude of expected floods would doubtless show harmonic variation, with nodes and slight inclinations as well as steep slopes. If the period of observation includes only one of the phases, there is a probability of a large error. Therein is the most potent argument for a physical examination of the watershed, channel, and modifying influences existing or to be provided.

As an example, the City of Pueblo, Colo., and the Denver and Rio Grande Railroad Company previous to 1921 had made ample provision against maximum recorded, or expected floods, based on the average annual crests, or on the possibility of local storms of the cloud-burst type affecting small tributaries

concurrently; but the demonstrations of June, 1921, placed this problem in another category. At that time (Fig. 7), there was a sustained violent storm over an area in excess of 1 000 sq. miles for at least 10 hours.

While making provision for increased channel capacity at Pueblo, engineers were astonished to find that the flood might have been of nearly double severity had the storm center been located a few miles farther westward. For this and other reasons, it was recommended that the old channel with a capacity of 40 000 sec.-ft. (fully five times the average yearly maximum for the preceding 30 years) be replaced by one including a wide spillway, the combined capacity to be 175 000 sec.-ft. The actual observed peak was 105 000 sec.-ft.

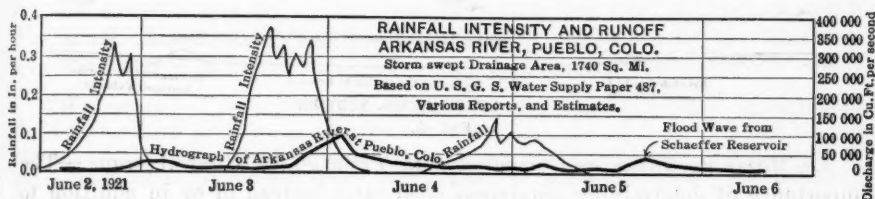


FIG. 7.

As another instance the spillway at the Lower Otay Dam, near San Diego, Calif., was overtaxed in January, 1916, the run-off being eight or ten times the quantity for which it was designed according to the known habits of the water-shed at that time. Such calamities emphasize the need for larger factors of safety to take care of these supposedly remote happenings.

4.—*Flood Height vs. Discharge.*—Where protective works are established or contemplated without storage the flood height is more important than the actual discharge passing the station. It is well established that a given stage of a rising flood in a silt-laden stream represents a greater flow than the same height during recession. In Fig. 8 are graphically shown some of the reasons for this phenomenon. It has been claimed for the Colorado River channel at Yuma, that for each foot of rise on the gauge it scours an equal depth from the bottom. Evidently after the peak has passed another cycle begins, the velocity being retarded, the silt burden depositing, the gradient and the hydraulic radius decreasing, thereby causing further retardation and deflection of currents laterally as if in quest of new material to transport. Although the tendency during the rising phase is to scour the channel bottom and induce central flow, the receding torrent is inclined toward lateral excursions. This often accounts for the fact that the main damage to alluvial farms occurs after the main crest has passed and the patrols have slackened their vigilance.

It remains an open question whether the tragedies too frequently recorded of passenger trains plunging from trestles into swollen streams are due to derailment or to inadequate inspection of the substructure. How is the track walker or inspector viewing the rising flood surface only 1 ft. or 2 ft. higher than the normal stream surface, to know that the channel may have scoured several feet deeper, possibly undermining the piers or piling?

While Sir William Willcocks was on a tour through America after his notable success at the Assuan Dam on the Nile, he stated that the 15-ft. gauge

height on a rising Tigris flood represents 180 000 sec.-ft. of discharge; the 20-ft. stage if at the peak represents only 120 000 sec.-ft., and the 15-ft. stage during recession corresponds to 90 000 sec.-ft.*

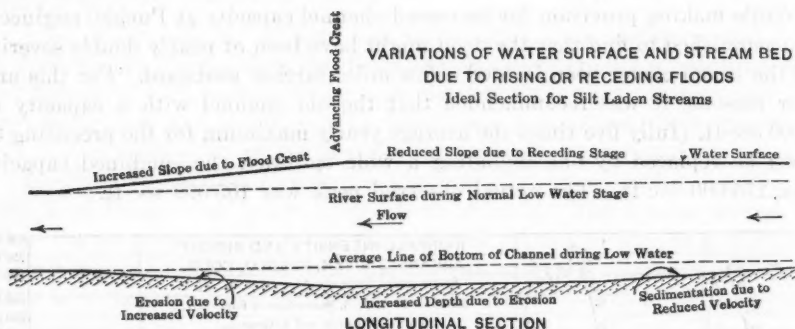


FIG. 8.

5.—*Maximum Stage as Compared with Twenty-four Hour Average.*—The importance of determining maximum flood rates instead of or in addition to the average for 24 hours, is well illustrated by the Pueblo flood (June 3, 1921), where the 24-hour rate was only 110% of the full channel capacity, while the maximum flow, lasting less than 1 hour but accountable for the main loss and damage, was more than twice as great.

In dealing with small areas it must be borne in mind that the maximum flood crest—sometimes coincident with the initial wave—and the return to normal flow may occur within a few hours. For this reason the collected data for Table 2 (Appendix I), and Plate VIII, deal with maximum rates wherever available.

6.—*The Wooded Channel.*—The regulating influence of great storage dams, such as those built by the U. S. Reclamation Service and enterprising irrigation districts, extends beyond the limits usually assigned. When they begin to function, the improvement of farms near the stream, where the alluvial soil is generally the most productive and at the same time most mellow and exposed to lateral erosion, increases the hazard of flood damage if maximum stage should recur. Even if the reservoir is full to the spillway floor at such a time it is effective, acting as a detention and regulating basin; and it has been conceded on some projects that the 6 to 10 ft. of storage depth above the spillway crest is as important as the entire capacity below that level. Additional depredations surely would have been made by the Colorado River in the Imperial Valley, California, had not the Roosevelt Dam been regulating the Salt River, the Gila's main tributary, when the Gila experienced a record flood in January, 1916, far exceeding any known discharge of the Colorado above Yuma.

If, however, a storm and flood period is protracted, the increased flow over the spillway or the necessity for the operation of emergency outlets to save the main structure, may overcome the natural equalizing effect and transmit the destructive flood stage to the lower river. In that case it would be discovered

* *Engineering Record*, June 27, 1914, p. 723.

that the channel capacity has decreased as compared with the years when unrestrained floods limited the growth of vegetation. The islands and bars formerly denuded would have such a well established covering of quick-growing trees as to resist and retard the maximum flood—perhaps even collecting driftwood barriers—forcing the water to seek higher levels. Carefully conducted gaugings* on typical channels have established the fact that for a wooded course Kutter's n increases to nearly double that obtaining for cleared areas.

7.—*Structural Design Limits and Flood Peaks.*—Granting that the economics of design would not justify doubling the expense to prevent interruption of bridge traffic two or three times a century, it still remains for the Engineering Profession to define just where the excess flow shall be accommodated. The U. S. Bureau of Public Roads anticipates conditions by the following specification:†

"*Waterway.*—The waterway shall be adequate for the passage of ordinary flood water and drift at a velocity not exceeding the average velocity of the stream in the vicinity of the bridge and shall be adequate for the passage of extreme flood water unless other provisions are made for same. By ordinary flood water is meant the stage that is likely to occur once in ten years."

To cross the numerous intermittent flood channels that are in service only a few hours or days each year, the Bureau and several States have adopted standard dips or spillways, consisting of undulations in the highway grade with suitable cut-off walls to prevent erosion or undermining. Comparative unit costs of waterway area provided by various types of drainage structures, indicate the great economy of the dip or spillway. Although it is conceded that this form of construction has its limitations, especially in connection with railroads and large drainage channels, it is believed that there is a broad field for its use in providing for the maximum flood stages that are beyond the economic range of other structures.

8.—*The Definite Function of Spillways.*—The moment a flood overtakes its definite channel is the beginning of either a minor episode or a great tragedy. If the rising surface is confined by inadequate artificial embankments until relief is secured only by the formation of crevasses, then the benefit sought by the levee system is more than nullified by the destructive velocity of an impounded flood suddenly released. It would be far better to allow the gradual overflow and less violent inundation of the adjacent lands; perhaps the enrichment of the soil due to an occasional quiet sedimentation would go far toward repairing the loss and inconvenience, if only the land owner has planned his crops and plantings, his permanent improvements, and his pastures for livestock so as to insure their safety or the minimum damage from such an expected overflow.

Sir William Willcocks, already quoted, evidently gave his own views of what should be done to control the Mississippi River more effectively, when he described the best practice of the ancients. He was altogether appreciative of the wonderfully effective work of the Corps of Engineers, U. S. Army, and of

* *Engineering News-Record*, October 12, 1922, p. 599.

† Superstructures, General Clauses, Standard Specifications, U. S. Bureau of Public Roads, as revised July, 1921.

the civilian personnel, and reiterated the idea that an occasional relief through crevasses is inevitable unless some form of multiple spillway along one bank or both shall begin to function automatically whenever the flood stage passes the limit for which ample provision has been made.

The alarm following the moderate but protracted floods of 1922 that passed the Arkansas Junction and registered the greatest flow ever recorded at Carrollton, La., is really a commendation for effective construction and maintenance of a great levee system, and gives point to the argument of Sir William Willcocks for automatic spillways, regulated overflow channels, and valley storage. Examinations of the actual projects on the Mississippi will disclose that all the foregoing features are contemplated in some degree; but where a stage higher than 50 ft. continues for 50 days, as occurred even at Cairo during that year, the use of farm lands for storage almost amounts to their abandonment.

Applying the principles herein enumerated to projects under civilian control, it would seem like a measure of safety to compute the profile of an assumed or estimated flood crest, which should be some multiple of the present channel capacity, in many cases double or treble. If the resulting lateral flow would overtax structures under highways, railroad embankments, or other barriers, the question would be to determine the possibility of their sudden failure, and the changes required to provide a safe automatic discharge with minimum damage.

When destructive floods occur at other times than during the planting or growing season and consist of sharp crests carrying much sediment, there are strong arguments for not only allowing but insuring the action of wooded channel spillways, thus providing against scouring velocities. The recurrent quiet sedimentation on the overflowed lands would add fertility and cause them to rise at least proportionately with the river delta.

REVIEW OF FLOOD PROBLEMS DESIRABLE

As a result of floods during recent decades and especially during 1923, a re-appraisal of the attendant dangers should be made for many of the river channels.

After a flood the stricken community is confronted with the problem of abandoning the inundated area in whole or in part, or of insuring it against future damage. This latter solution may be accomplished either by positive control—that is, regulation of maximum crests—or by adequate protective works. Conditions may favor (1) regulation by detention basins, ordinary reservoirs, or those purposely located to favor outflow through underground channels; (2) diffusion of the water as by spreading it over detrital cones, gravel bars, or coarse sandy areas to replenish underground storage; (3) establishing timber cribs or other barriers as overfall dams to break the continuous gradient, to check a rolling initial wave and temporarily arrest the heaviest burden; (4) planting willow and timber copses to entrap sediment; or (5) diffusing the surplus by spilling it into pasture or woodlands. All these methods are based on retardation. The case may demand the widening, rectification, and clearing of the channel to promote accelerated flow, or it may

warrant a combination of the two general methods—truly, a wide field for investigation. The number of possible solutions of such problems forms a maze through which the proper path is difficult to discover, requiring the best talent and the most mature judgment that experience, and special training, can develop.

NEED FOR UNIFIED CONTROL

So complex and interlocked are the community interests to be served in the control of streams that there is scant opportunity for the rough and ready, independent, private initiative which pioneers had to develop in order to survive. According to present paternalistic policies, it seems imperative that this control become a State or municipal function for small streams, even as it is now administered under Federal jurisdiction for navigable rivers.

The shortcomings of the present methods are forcibly illustrated by a case that occurred during 1923. When a torrential flood had started to ravel the protective rip-rap and eat into city property, a mass meeting was called, at which the State Engineering Department, represented in an advisory capacity, recommended the anchoring of newly cut trees, with branches intact, as current deflectors and retards, and the placing of brush and rock revetment among hurriedly driven piles and stakes. In earlier days this very plan would have been carried out and reported as accomplished before the hour for mass meetings; but in the present case by the time each citizen had extolled his individual plan, the majority sentiment seemed to favor the use of rock because of its durable character. This disregard of sound engineering advice for wasteful emergency measures, resulted in the continued progress of erosion, with the loss of all the new rip-rap, several city lots and comfortable homes, an appeal to the Government for relief, and the acceptance of a plan furnished by a competent specialist requiring a timber pile jetty with brush and rock-filling to cover the scar and prevent further incision.

OPPORTUNITY FOR PUBLIC SERVICE

The possibilities of the present situation must appeal to all members of the profession who are experienced and competent in flood protection, that is, the privilege of contributing to community welfare in determining whether great hazards do exist and what special protective measures are required. If thereby local officials are awakened to the reasonable needs, and relief projects are initiated, one step toward safety will have been accomplished; the danger zone of floods will be known and provision will be made for an emergency. If even a small percentage of the annual disasters should be averted, or the loss and damage reduced by wise preparation, the cost and labor involved in such studies will be fully justified.

CONCLUSION

In this paper, the writer does not assume to solve any particular problem, but merely lists certain data and considerations that are often left unrelated or neglected entirely. His purpose will be accomplished only when the essential suggestions herein recorded, or to be brought forth during the discussion, have

resulted in a definite campaign to determine flood dangers and the advisable measures for control. The most positive step yet taken seems to be the studies involving the expected run-off for California streams made by the State Engineering Department.*

Specific acknowledgments have been made wherever practicable in the paper. In addition, the writer has obtained invaluable information from the published writings of the many eminent hydraulic engineers of the United States, most of them members of this Society.

Special appreciation is due those who have so generously assisted and encouraged this work, notably Government engineers and officials, State, municipal, and college officers, engineers in private practice, and associates.

* Reference No. 70, Table 2 (Appendix I).

APPENDIX I

TABLE 2.—MAXIMUM OBSERVED FLOOD DISCHARGE RATES AND EXPECTED RUN-OFF FROM VARIOUS DRAINAGE BASINS ARRANGED ACCORDING TO AREA.

No.	River or tributary and location.	Drainage area, in square miles.	RUN OFF, IN SECOND- FEET PER SQUARE MILE.				Main authority, see Appendix II.	Location.*	
			Observed.	Expected peaks.		Frequent.			Rare.
				Maximum and date.					
1	Wailua, South Fork, Oahu, Hawaii.....	0.23	1 955	Jan., 1916			78	H.	
2	Beacon Creek, near Fishkill, N. Y.....	0.25	3 200 1897			87	N.E.	
3	Bull's Run, Long Level, Pa.....	0.58	4 170	July, 1914			71	N.E.	
4	Docker's Hollow, North Braddock, Pa.....	0.60	4 000	June, 1917			71	N.E.	
5	Mann's Run, Creswell Station, Pa.....	0.67	2 540	July, 1914			71	N.E.	
6	Manoa, West Branch, Oahu, Hawaii.....	1.00	3 250	Jan., 1916			78	H.	
7	Manoa, East Branch, Oahu, Hawaii.....	1.10	2 810	Jan., 1921			78	H.	
8	China Wash, near Hurricane, Utah.....	1.10	500	Aug., 1916	300	600	74	R.M.	
9	Budlong Creek, near Utica, N. Y.....	1.13	120 1904			2	N.E.	
10	Sylvan Glen Creek, New Hartford, N. Y.....	1.18	57 1904			71	N.E.	
11	Mad Creek, Leroy, N. Y.....	1.5	2 300			71	N.E.	
12	Nuuanu Reservoir No. 4, Hawaii.....	1.5	1 600	Feb., 1907			78	H.	
13	Pequest, Hunt's Pond, N. J.....	1.7	25 1904			71	N.E.	
14	Green Branch, Bridgeville, Pa.....	1.7	1 595	July, 1914			83	N.E.	
15	South Arroyo, near Pueblo, Colo.....	1.8	1 060	June, 1921			68	R.M.	
16	Provo (Head-waters), Wall Lake, Utah.....	2.0	350	June, 1911	200	400	74	G.B.	
17	Cherryvale Creek, Cherryvale, Kans.....	2.0	930			31	M.V.	
18	Indian Run, Letort, Pa.....	2.1	1 930			71	N.E.	
19	Canadachly Branch, East Prospect, Pa.....	2.2	1 630	July, 1914			71	N.E.	
20	Rio Grande, near Culebra, Panama.....	2.4	161			32	P.	
21	Colvin Run, Grindstone, Pa.....	2.7	480	July, 1912			91	N.E.	
22	San Pedro Creek, San Antonio, Tex.....	2.7	750	Sept., 1921			67	M.V.	
23	Canodochly Creek, Branch, Pa.....	3.2	1 120	July, 1914			80	N.E.	
24	Starch Factory Creek, N. Y.....	3.4	209	Sept., 1905	130		71	N.E.	
25	Provo, Washington Lake, Utah.....	3.4	300	June, 1911	200	350	74	G.B.	
26	Estanguela, near Monterey, Mexico.....	3.5	825	Aug., 1909	300	700	35	M.	
27	Switzer Canyon, San Diego, Calif.....	3.5	188	Jan., 1916			66	S.W.	
28	Nuuanu, U. S. Geological Survey Gauge, Hawaii.....	3.5	952	Jan., 1916			78	H.	
29	Venison Branch, near Mulga, Ala.....	3.9	53			11	S.E.	
30	North Canyon, near Centerville, Utah.....	4.0	450	Aug., 1923	50	500	74	G.B.	
31	Reel's Creek, near Dearfield, N. Y.....	4.4	670	55		28	N.E.	
32	Kaukonahua, Upper Dame, Hawaii.....	4.5	1 605	Jan., 1916			78	H.	
33	Mill Brook, Sherbourne, N. Y.....	5.0	262 1905			91	N.E.	
34	Hall's Gulch, Boise, Idaho.....	5.0	1 000	July, 1913			91	N.E.	
35	Breakbeck Run, Bullskin Tp., Pa.....	5.2	310	May, 1902			91	N.E.	
36	Kaneohe, Pali, Oahu, Hawaii.....	5.3	2 070	Jan., 1916			78	H.	
37	Brush Creek, Jeanette, Pa.....	6.0	500	July, 1903			91	N.E.	
38	Skinner Creek, Mannsville, N. Y.....	6.4	124	July, 1891			71	N.E.	
39	Cold Spring, Brook, Mass.....	6.4	48 1886			71	N.E.	
40	Blue Ribbon Creek, Pueblo, Colo.....	6.7	1 360	June, 1921			68	R.M.	
41	Honey Creek, East Fork, New Carlisle, Pa.....	6.7	2 210	July, 1918			71	N.E.	
42	Farmington Creek, Farmington, Utah.....	7.0	350	Aug., 1923			74	G.B.	
43	Templeton Gap, Colorado Springs, Colo.....	7.1	862	May, 1922			76	R.M.	
44	Cameron Arroyo, near Pueblo, Colo.....	7.3	1 900	June, 1921			68	R.M.	
45	Camp Branch, Ensley, Ala.....	7.4	68.8 1909			11	S.E.	
46	Osteen Arroyo, near Pueblo, Colo.....	7.8	1 160	June, 1921			68	R.M.	
47	Croton, South Branch, N. Y.....	7.8	74 1869			71	N.E.	
48	Burgoon's Run, Pennsylvania.....	8.0	400	May, 1894			91	N.E.	
49	Independence Creek, California.....	8.5	34	27		69	G.B.	
50	Israel, Jefferson Highlands, N. H.....	8.7	63	41		69	N.E.	
51	Arroyo, Indiole, N. Mex.....	8.9	1 105	July, 1915			82	M.V.	
52	Fred Rohr Gulch, Pueblo, Colo.....	9.3	104	June, 1921			68	R.M.	

* Locations as given in last column of Table 2, are abbreviated, as follows: Northeastern United States, N.E.; Southeastern, S.E.; Southwestern, S.W.; Northwestern, N.W.; Mississippi Valley, M.V.; Rocky Mountains, R.M.; Great Basin, G.B.; Alaska, A.; Canada, C.; Hawaii, H.; Mexico, M.; Panama, P.; and foreign, F.

TABLE 2.—(Continued).

No.	River or tributary and location.	Drainage area, in square miles.	RUN-OFF, IN SECOND-FEET PER SQUARE MILE.				Main authority, see Appendix II.	Location.*	
			Observed.	Expected peaks.		Frequent.			Rare.
				Maximum and date.					
53	Woodhall, Reservoir, Herkimer, N. Y.	9.4	78	1869	71	N.E.	
54	Mill Brook, near Sherbourne, N. Y.	9.4	241	Sept.,	1905	1	N.E.	
55	Dry Run, Ohio.	9.8	460	1912	69	M.V.	
56	Baker Creek, Baker, Nev.	10.0	17	1914	72	G.B.	
57	Spring Creek, Harrisburg, Pa.	11.6	259	Feb.,	1908	69	N.E.	
58	Honey Creek, East Fork, New Carlisle, Pa.	11.8	1 285	71	N.E.	
59	Boulder Creek, near Julian, Calif.	12.0	217	Jan.,	1916	66	S.W.	
60	Mill Creek, Erie, Pa.	12.9	1 000	Aug.,	1915	91	N.E.	
61	Goodyear Creek, Goodyear Bar, Calif.	12.2	97	3	S.W.	
62	Stony Brook, Boston, Mass.	12.7	121	72	N.E.	
63	Tomer, above Reservoir, Holyoke, Mass.	13.0	52	33	69	N.E.	
64	Manhan River, Holyoke, Mass.	13.0	182	Feb.,	1900	91	N.E.	
65	Connoquenessing Creek, Oakland Pa.	13.6	315	Aug.,	1908	91	N.E.	
66	Chandler Creek, near Pueblo, Colo.	13.6	118	June,	1921	68	R.M.	
67	Canodochly Creek near Mouth, Pa.	13.9	359	July,	1914	80	N.E.	
† 68	Great River, Westfield, Mass.	14.0	71	72	N.E.	
69	Panther Creek, Iowa.	14.0	520	June,	1905	91	M.V.	
70	Grand Central, below Forks, Alaska.	14.6	100	1906	72	A.	
71	Rocky Creek, near Ellisville, Miss.	15.0	1 110	May,	1882	91	S.E.	
72	North Arroyo, near Pueblo, Colo.	15.6	619	June,	1921	68	R.M.	
73	Rio des Perca, Clayton Road, St. Louis, Mo.	15.6	410	Aug.,	1915	81	M.V.	
74	Broad Creek, Millgreen, Md.	16.4	31	17	69	N.E.	
75	Arroyo Seco, near Pasadena, Calif.	16.4	192	Jan.,	1916	66	S.W.	
76	Williamstown, Williamstown, N. Y.	16.5	34	72	N.E.	
77	Alazon Creek, San Antonio, Tex.	17.1	1 515	Sept.,	1921	67	M.V.	
78	Waiawa Waste, Oahu, Hawaii	17.5	343	Jan.,	1910	78	H.	
79	Ford Creek, Augusta, Mont.	18.0	40	22	69	R.M.	
80	Little Devil's Creek, Iowa.	19.0	560	June,	1905	91	M.V.	
81	Yuba, Bowman Dam, Calif.	19.0	32	72	S.W.	
82	City Creek, Salt Lake City, Utah.	19.2	8	6	50	69	G.B.
83	Rush Creek, near Pueblo, Colo.	19.6	238	June,	1921	68	R.M.	
84	Martinez Creek, San Antonio, Tex.	19.6	1 223	71	M.V.	
85	Ridley Creek, near Philadelphia, Pa.	20.0	750	Aug.,	1848	77	
86	Chase Creek of Gila River, Arizona	20.0	647	Dec.,	1906	91	R.M.	
87	Willow Creek, near Heppner, Ore.	20.0	1 800	June,	1903	19	N.W.	
88	Croton, West Branch, N. Y.	20.5	54	1874	72	N.E.	
89	Beaverdam Creek, Altmar, N. Y.	20.7	111	72	N.E.	
90	Israel, below South Branch, N. H.	21.2	49	34	69	N.E.	
91	Mill Creek, Salt Lake City, Utah	21.3	6	4	30	90	G.B.
92	Brush Hollow Creek, Pueblo, Colo.	21.9	243	June,	1921	68	M.V.	
93	Crum Creek, near Philadelphia, Pa.	22.0	410	Aug.,	1848	77	N.E.	
94	Cane Creek, Bakersville, N. C.	22.0	1 341	May,	1901	34	S.E.	
95	Penitencia Creek, San Jose, Calif.	22.0	40	59	70	S.W.
96	Coal Creek, near Pueblo, Colo.	22.3	167	June,	1921	68	R.M.	
97	Dry Run, Decorah, Iowa	22.3	720	Mar.,	1915	91	M.V.	
98	Ritchie Gulch, near Pueblo, Colo.	22.6	41	June,	1921	68	R.M.	
99	Trout Brook Centerville, N. Y.	23.0	51	72	N.E.	
100	Wailua, near Lihue, Kanai, Hawaii.	23.0	1 960	Jan.,	1916	78	H.	
101	Rio des Perca, near St. Louis, Mo.	23.8	256	Aug.,	1915	81	M.V.	
102	Apache Creek, San Antonio, Tex.	23.8	948	Sept.,	1921	67	M.V.	
103	Mill, North Fork, Pinkbed, N. C.	24.0	49	June,	1921	32	69	S.E.	
104	Six-Mile Creek, near Pueblo, Colo.	24.6	77	68	R.M.	
105	Trout Brook, Brooksport, N. Y.	25.0	158	69	N.E.	
106	Pequonnock, Bridgeport, Conn	25.0	157	July,	1905	1	N.E.	
107	Middle Creek, Bozeman, Mont.	26.0	31	15	50	69	R.M.
108	Smith Creek, Augusta, Mont.	26.0	31	23	60	69	R.M.
109	Olmos Creek, San Antonio, Tex.	26.0	948	Sept.,	1921	67	M.V.	
110	Boggs Creek, near Pueblo, Colo.	26	582	June,	1921	68	R.M.	

† Maximum daily flow.

TABLE 2.—(Continued).

No.	River or tributary and location.	Drainage area, in square miles.	RUN-OFF, IN SECOND- FEET PER SQUARE MILE.				Main authority, see Appendix H.	Location.*
			Observed.	Expected peaks.				
				Maximum and date.	Frequent.	Rare.		
111	Spring Creek, above Dayton, Ohio.....	27.0	210 , Mar., 1913	91	M.V.	
112	Bear Grass Creek, Louisville, Ky.....	27.5	100 , , , 1908	40	M.V.	
113	Donnels Creek, above Dayton, Ohio.....	27.5	147 , Mar., 1913	91	M.V.	
114	Little Cottonwood, Salt Lake City, Utah..	27.7	25 , June, 1912	13	40	87	G.B.	
115	Cottonwood Creek, near Fresno, Calif....	28.0	20	26	70	S.W.	
116	Wantuppa Lake, Fall River, Mass.....	28.5	72 , , , 1875	72	N.E.	
117	Emigration Creek, Salt Lake City, Utah..	29.0	6 , , ,	8	20	74	G.B.	
118	American Fork, above Power Plant, Utah..	30.0	12 , May, 1914	10	50	87	G.B.	
119	Pinal Creek, Globe, Ariz.....	30.0	440 , Aug., 1904	91	R.M.	
120	Arroyo Secco, near Pasadena, Calif.....	30.5	374 , May, 1914	73	S.W.	
121	Donner Creek, near Truckee, Calif.....	30.5	32 , , ,	19	34	69	G.B.	
122	Moosa Creek, near Bonsall, Calif.....	31.0	269 , Jan., 1916	66	S.W.	
123	Pequest, Huntsville, N. J.....	31.4	19 , , ,	72	N.E.	
124	Ink's Creek, near Red Bluff, Calif.....	34.0	120	153	70	S.W.	
125	San Antonio, San Antonio, Tex.....	34.3	691 , Sept., 1921	67	M.V.	
126	Peck's Creek, near Pueblo, Colo.....	34.4	564 , June, 1921	68	R.M.	
127	Sawkill, near Mouth, N. Y.....	35.0	229 , Apr., 1895	69	N.E.	
128	Chalk Creek, Fillmore, Utah.....	38.0	13 , , , 1914	72	G.B.	
129	Whippany River, Whippany, N. J.....	38.0	94 , , , 1896	24	N.E.	
130	Turtle Creek, above Dayton, Ohio.....	35.0	175 , Mar., 1913	91	M.V.	
131	San Francisquito Creek, California.....	38.0	51	62	70	S.W.	
132	Lake Roland, Maryland.....	39.0	230 , , , 1868	91	N.E.	
133	Cuyadutta, Johnstown, N. Y.....	40.0	72 , , , 1896	71	N.E.	
134	Mill, South Fork, Sittou, N. C.....	40.5	51 , Jan., 1916	39	69	N.E.	
135	Red Creek, near Pueblo, Colo.....	40.6	22 , June, 1921	68	R.M.	
136	San Pablo Creek, Berkeley, Calif.....	41.0	38	50	70	S.W.	
137	Little Johns Creek, near Stockton, Calif.	41.0	70	107	70	S.W.	
138	Bassie Creek, Donedroid, N. Y.....	41.0	81 , , ,	27	N.E.	
139	Westfield Little, Blandford, Mass.....	43.0	50 , , ,	34	69	N.E.	
140	Little Gun Powder, Bel Air, Md.....	43.0	37 , , ,	20	69	N.E.	
141	Sweetwater River, Descano, Calif.....	43.7	326 , Jan., 1916	66	S.W.	
142	Elkhorn Creek, Keystone, W. Va.....	44.0	1 363 , June, 1901	84	M.V.	
143	Cameron Creek, Hurley, N. Mex.....	44.0	135 , Aug., 1913	91	M.V.	
144	San Leandro Creek, Oakland, Calif.....	44.0	38	50	70	S.W.	
145	Six Mile Creek, Ithaca, N. Y.....	46.0	185 , June, 1905	1	N.E.	
146	San Pedro Creek, below Apache Creek, Texas.....	46.5	698 , Sept., 1921	67	M.V.	
147	Wallowa, Joseph, Ore.....	47.0	16 , , ,	14	69	N.W.	
148	Dog River, Northfield, Vt.....	47.0	72 , , , 1913	72	N.E.	
149	West Canada Creek, Mott's Dam, N. Y.....	47.5	34 , , ,	72	N.E.	
150	Dry Creek, near Fresno, Calif.....	48.0	42	51	70	S.W.	
151	Pine Creek, Paris, Tex.....	48.0	410 , May, 1920	91	M.V.	
152	Prosser Creek, Hobart Mills, Calif.....	48.0	19 , , ,	11	69	S.W.	
153	Darby Creek, near Philadelphia, Pa.....	48.0	580 , Aug., 1843	77	N.E.	
154	Turkey Creek, near Pueblo, Colo.....	48.0	188 , June, 1921	68	R.M.	
155	Big Cottonwood, Salt Lake City, Utah.....	48.5	17 , , ,	9	30	69	G.B.	
156	Conecmaugh Branch (Johnstown flood), Pa.	48.6	206 , May, 1889	69	N.E.	
157	Parley's Creek, Salt Lake City, Utah.....	50.1	6 , , ,	3	20	69	G.B.	
158	Sangquilt Creek, New York Mills, N. Y.....	51.5	53 , , ,	71	N.E.	
159	Kosh Creek, near Henderson, Calif.....	51.9	44 , , ,	8	S.W.	
160	Cottonwood Creek, North Fork, Ono, Calif.	52.0	78 , , ,	8	S.W.	
161	Lost Creek, above Dayton, Ohio.....	52.0	571 , Mar., 1913	91	M.V.	
162	Guadalupe River, California.....	52.0	55	75	70	S.W.	
163	Rockaway, Dover, N. J.....	52.5	53 , , ,	71	N.E.	
164	Santa Ysabel Creek, Mesa Grande, Calif..	53.4	395 , Jan., 1916	66	S.W.	
165	Tawawa Creek, above Dayton, Ohio.....	54.0	239 , Mar., 1913	91	M.V.	
166	Yakima, Martin, Wash.....	56.0	11 , , ,	69	N.W.	
167	Santa Maria Creek, Ramona, Calif.....	57.3	125 , Jan., 1916	66	S.W.	
168	Oneida Creek, Kenwood, N. Y.....	59.0	41 , , , 1890	71	N.E.	
169	Rock Creek, near Pueblo, Colo.....	59.0	913 , June, 1921	68	R.M.	
170	Wildcat Creek Group, Stockton, Calif.....	59.0	70	97	70	S.W.	

TABLE 2.—(Continued).

No.	River or tributary, and location.	Drainage area, in square inches.	RUN-OFF, IN SECOND-FOOT PER SQUARE MILE.				Main authority, see Appendix II.	Location.*	
			Observed.	Expected peaks.		Frequent.			Rare.
				Maximum and date.					
171	Lytle Creek, San Bernardino, Calif.....	60.0	267	Jan., 1916			66	S.W.	
172	Flat, Rhode Island.....	61.0	120	1848			72	N.E.	
173	Camden Creek, Camden, N. J.....	61.4	24	1889			72	N.E.	
174	Peguannock, Macopin, N. J.....	62.0	91				20	N.E.	
175	Chester Creek, near Philadelphia, Pa.....	62.0	1 000	Aug., 1843			77	N.E.	
176	Los Angeles, near Los Angeles, Calif.....	62.0	117	Jan., 1916			66	S.W.	
177	Nine-Mile Creek, Slittville, N. Y.....	62.6	125	1898			72	N.E.	
178	Kachess, near Easton, Wash.....	63.0	36		24		69	N.W.	
179	Naches, Niles, Wash.....	63.6	341		137		69	N.W.	
180	Wissahickon Creek, Philadelphia, Pa.....	64.6	43	1898			72	N.E.	
181	Eight-Mile Creek, near Pueblo, Colo.....	65.0	154	June, 1921			68	R.M.	
182	Ludlow Creek, above Dayton, Ohio.....	65.0	266	Mar., 1913			91	M.V.	
183	American Fork, American Fork, Utah.....	65.8	13		7		69	G.B.	
184	San Jacinto, at Hemet Reservoir, Calif.....	65.8	145	Jan., 1916			66	S.W.	
185	Owens Creek Group, near Merced, Calif.....	66.0			33	44	70	S.W.	
186	Daulton Creek Group, California.....	66.0			40	51	70	S.W.	
†187	Carson, West Fork, Woodfords, Calif.....	67.0	23	May, 1906			70	S.W.	
188	Oak Creek, near Pueblo, Colo.....	68.0	41	June, 1921			68	R.M.	
189	Sandy Creek, Allendale, N. Y.....	68.4	89	1891			72	N.E.	
190	Jamul Creek, near Otay, Calif.....	69.8	259	Jan., 1916			66	S.W.	
191	Bear Creek, near Madera, Calif.....	71.0			36	50	70	S.W.	
192	Dutchman Creek Group, Madera, Calif.....	72.0			40	54	70	S.W.	
193	Laramie Reservoir Outlet, Laramie, Wyo.....	72.0	97	Mar., 1913			38	M.V.	
194	Butte Creek, Butte Valley, Calif.....	73.0	23		16		8	S.W.	
195	Brushy Creek, Round Rock, Tex.....	74.7	462	Sept., 1921			67	M.V.	
196	San Vincent Creek, Foster, Calif.....	74.9	248	1916			72	S.W.	
197	Mission Creek Group, San Jose, Calif.....	77.0			43	59	70	S.W.	
198	Rock Creek, Washington, D. C.....	77.5	126				72	N.E.	
199	Sonoma Creek Group, Sonoma, Calif.....	78.0			52	66	70	S.W.	
200	Sudbury, Farmington, Mass.....	78.0	41	1897			72	N.E.	
201	Peguannock, Pompton, N. J.....	78.0	56	1902			72	N.E.	
202	Hockanum River, Conn.....	79.0	78				72	N.E.	
203	Chittenango Creek, Chittenango, N. Y.....	79.0	24				69	N.E.	
204	Dry Creek, near Auburn, Calif.....	79.0			100	124	70	S.W.	
205	Bear Creek, near Cape Mendocino, Calif.....	82.0			130	162	70	N.W.	
206	Claremont Creek Group, Claremont, Calif.....	83.0			40	50	70	S.W.	
207	Lagunitas Creek, near Sansalito, Calif.....	84.0			32	42	70	S.W.	
208	San Mateo Creek Group, California.....	84.0			44	58	70	S.W.	
209	Kraiz Gamepa Outlet, Salmon, Alaska.....	84.0	51	1902			72	A.	
210	Nashua River, Massachusetts.....	84.5	71	1850			72	N.E.	
211	San Antonio, below San Pedro Creek, Texas.....	85.0	499	Sept., 1921			67	M.V.	
212	Roach, Roach, Me.....	85.0	23		19		69	N.E.	
213	Dry Creek, near Pueblo, Colo.....	86.0	283	June, 1921			68	R.M.	
214	Willow Creek, near Augusta, Mont.....	90.0	42	May, 1917			68	R.M.	
215	Gallinas, Hot Springs, N. Mex.....	90.0	129	Sept., 1904			68	M.V.	
216	Putah Creek, near Guenoc, Calif.....	91.0	270	Mar., 1904			91	S.W.	
217	Independence Creek, Crandall, N. Y.....	93.2	66	1869			71	N.E.	
218	Boulder, Bruffeys, Mont.....	94.0	17		16		69	R.M.	
219	Asay Creek, Hatch, Utah.....	96.0	17	1913			72	G.B.	
220	North Fork Creek, Brookville Pa.....	97.0	124	July, 1912			91	N.E.	
221	Malabu Creek, Cabasas, Calif.....	97.0	70		27		69	S.W.	
222	Yokohi Creek Group, near Tulare, Calif.....	98.0			30	38	70	S.W.	
223	Otay, Lower Otay Dam, Calif.....	98.6	379	Jan., 1916	40	300	70	S.W.	
224	Churn Creek Group, California.....	100.0	100		100	133	70	S.W.	
225	Passaic, Chatham, N. J.....	101.0	30		20		69	N.E.	
226	Wenague, Pompton, N. J.....	101.0	84			24	24	N.E.	
227	Deer River, Deer River, N. Y.....	101.0	78	1869			72	N.E.	
228	Swift Current, near Babb, Mont.....	101.0	45		22		69	R.M.	
229	Tolrickon Creek, Mt. Pleasant, Pa.....	102.0	138	1894	41		72	N.E.	
230	Little Stony Creek, near Ladaga, Calif.....	102.0	69				8	S.W.	

† Maximum daily flow.

TABLE 2.—(Continued).

No.	River or tributary, and location.	Drainage area, in square miles.	RUN-OFF, IN SECOND-FOOT PER SQUARE MILE.				Main authority, see Appendix II.	Location.*	
			Observed.	Expected peaks.		Frequent.			Rare.
				Maximum and date.					
†231	Mariposa Creek, near Merced, Calif.	108			43	56	70	S.W.	
232	Fish Creek, East Branch, Point Rocks, N. Y.	104	805	1897			72	N.E.	
233	Valley, Tomotia, N. C.	106	97		47		69	N.E.	
234	San Jacinto River, near San Jacinto, Calif.	108	278	Jan., 1916			66	S.W.	
235	Onondago Creek, Syracuse, N. Y.	108	30	1913			37	N.E.	
236	Nashua, Massachusetts.	109	104	1848			72	N.E.	
†237	Red Bank Creek Group, near Tulare, Calif.	109			67	89	70	S.W.	
238	Whiterocks, Whiterocks, Utah.	110	10			6	69	G.B.	
†239	Deer Creek, near Red Bluff, Calif.	110			32	38	70	S.W.	
240	Sandy Creek, North Branch, Adams, N. Y.	110	67	1897			72	N.E.	
241	Santa Ysabel Creek, near Ramona, Calif.	110	254	Jan., 1916			66	S.W.	
242	Lewiston Reservoir, Outlet, Ohio.	111	58				38	M.V.	
243	Sweetwater River, near Dehesa, Calif.	112	217	Jan., 1916			66	S.W.	
244	Rockaway, Boonton, N. J.	118	49				24	N.E.	
245	Ohanapecosh, near Lewis, Wash.	116	65	1909			72	N.W.	
246	Laurel Hill Creek, Confluence, Pa.	118	42		25		69	N.E.	
247	Scantic, North Branch, Conn.	118	52				72	N.E.	
248	Ramapo, Mahwah, N. J.	118	105	1903			72	N.E.	
249	Clear Creek, Buffalo, Wyo.	118	7		6		69	R.M.	
250	Antelope Valley Group, near Los Angeles, Calif.	119			54	87	70	S.W.	
251	Los Gatos Creek, near Hanford, Calif.	119			27	38	70	S.W.	
252	Cottonwood Creek, Mooreana, Calif.	119	128	Jan., 1916			66	S.W.	
253	Hobble Creek, Springville, Utah.	120	7		3		69	G.B.	
254	Los Gatos Creek Group, near Redwood, Calif.	121			46	62	70	S.W.	
255	Martells Creek Group, near Stockton, Calif.	122			70	97	70	S.W.	
256	Suisun Creek Group, near Suisun, Calif.	125			43	52	70	S.W.	
257	Rockaway, Boonton, N. J.	125	22	1902			72	N.E.	
258	South Boulder, Marshall, Colo.	125	9		5		69	R.M.	
259	Santa Ysabel Creek, near San Diego, Calif.	126			80	154	70	S.W.	
260	Laurel Hill Creek, Confluence, Pa.	126	40				22		
261	Patuxent, Laurel, Md.	127	40	1915			72	N.E.	
262	Walnut Canyon, near Flagstaff, Ariz.	128	82	Sept., 1923			74	S.W.	
263	Fresno River, near Knowles, Calif.	134	34	Feb., 1917			66	S.W.	
264	Bear Creek Group.	137			100	131	70	S.W.	
265	Cedar, Ravensdale, Wash.	138	91		35		90	N.W.	
266	San Antonio Creek, near Santa Barbara, Calif.	138			58	73	70	S.W.	
267	Petaluma Creek Group, near Petaluma, Calif.	139			46	57	70	S.W.	
268	Union River (West Branch), Maine.	140	50	Apr., 1923	15		88	N.E.	
269	Neshaminy Creek, below Forks, Pa.	140	98	1894	33		72	N.E.	
270	Dead, Forestville, Mich.	142	17		16		69	M.V.	
271	Devil's Creek, near Viele, Iowa.	143	600+	June, 1905			91	M.V.	
272	Oriskany Creek, Oriskany, N. Y.	144	51				27	N.E.	
273	Turtle Creek, East Pittsburgh, Pa.	146	64				26	N.E.	
274	Musconetcong, Bloomsburg, N. Y.	146	19		13		69	N.E.	
275	Salado Creek, Salado, Tex.	148	966	Sept., 1921			67	M.V.	
276	Salmon Creek, Malott, Wash.	152	4		3		69	N.W.	
277	Perkiomen Creek, Frederick, Pa.	152	116	1894	35		72	N.E.	
278	Mohawk, Ridge Mills, N. Y.	153	46				72	N.E.	
279	Bollinas Creek Group, Bollinas Bay, Calif.	158			20	25	70	S.W.	
280	Soques, Demorest, Ga.	158	56		36		69	S.E.	
281	Mora River, below Mora, N. Mex.	159	140	Sept., 1904			2	M.V.	
282	Gunpowder Falls, Glencoe, Ind.	160	35		34		69	N.E.	
283	Ramapo, Pompton, N. J.	160	66	1832			24	N.E.	
284	Kinzua Creek, Dew Drop, Pa.	162	20				22	N.E.	
285	Weber, Oakley, Utah.	163	25		17		69	G.B.	
286	Mono Lake Group, Calif.	166			20	25	70	G.B.	
287	Little Truckee, Starr, Calif.	166	11		9		69	G.B.	
288	Los Angeles River, Tributary, Calif.	167			70	130	70	S.W.	
289	Fish Creek, East Branch, Taberg, N. Y.	169	65	1913			37	N.E.	
290	Cedar, Ravensdale, Wash.	170	64		27		69	N.W.	

† Maximum daily flow.

TABLE 2.—(Continued).

No.	River or tributary, and location.	Drainage area, in square miles.	RUN-OFF, IN SECOND-FEET PER SQUARE MILE.				Main authority, see Appendix II.	Location.*
			Observed.	Expected peaks.				
				Maximum and date.	Frequent.	Rare.		
291	Burns Creek Group, Merced, Calif.....	171			50	66	70	S.W.
292	Sweetwater River, Jamacho, Calif.....	172	250	, 1916			66	S.W.
293	Hardscrabble Creek, Pueblo, Colo.....	173	19	, June, 1921			68	R.M.
294	St. Mary, near Babb, Mont.....	177	45		19		69	R.M.
295	Backbone Creek, Red Bluff, Calif..	178			80	120	70	S.W.
296	Bear Creek, Morrison, Colo.....	180	48	, July, 1896			68	R.M.
297	Fish Creek, West Branch, McConnellsville, N. Y.....	187	33	, 1885			72	N.E.
298	Smoke Creek Group, near Honey Lake, Calif.....	188			10	11	70	G.B.
299	Santa Ana River, Mentone, Calif.....	189	154	, Jan., 1916			66	S.W.
300	San Diego River, Lakeside, Calif.....	189	201	, Jan., 1916			66	S.W.
301	Seekonk, Providence, R. I.....	190	57	, 1867			72	N.E.
302	Tallulah, Tallulah Falls, Ga.....	191	41			26	16	S.E.
303	Coyote River, Madrone, Calif.....	197	127	, Mar., 1911			70	S.W.
304	Fishkill Creek, at Glenham, N. Y.....	198	69				21	N.E.
305	Santa Ana River Tributaries, Mentone, Calif.....	199	146	, Jan., 1916			70	S.W.
306	Williams Fork, Sulphur Springs, Colo....	200	7		5		69	R.M.
307	Canyon Padre, Diablo Jct., Ariz.....	200	55	, Sept., 1923			74	R.M.
308	Mt. Diablo Creek, Martinez, Calif.....	200			47	63	70	S.W.
309	Cave Creek, Phoenix, Ariz.....	200	125	, Aug., 1921			91	S.W.
310	Limekiln Creek, Tulare, Calif.....	201			30	38	70	S.W.
311	Unadilla, New Berlin, N. Y.....	204	40	, 1905			67	N.E.
312	Cle Elum Lake, Roslyn, Wash.....	205	86		36		69	N.W.
313	Santa Yuez, Santa Barbara, Calif.....	207	46	, 1907			72	S.W.
314	San Diego River, San Diego, Calif.....	207	72			10	70	S.W.
315	Cantua Creek, Hanford, Calif.....	208			20	32	70	S.W.
316	San Luis Rey River, Mesa Grande, Calif..	209	280	, Jan., 1916			66	S.W.
317	St. Vrain Creek, Lyons, Colo.....	209	6		5		69	R.M.
318	Coon Creek, Sacramento, Calif.....	210			70	89	70	S.W.
319	Catskill Creek, South Cairo, N. Y.....	210	100	, 1901			27	N.E.
320	Mojave River, Victorville, Calif.....	211	64	, Mar., 1903	65	109	70	S.W.
321	French Creek, North Branch, Kimmeytown, Pa.....	212	43				22	N.E.
322	Beaver Creek, Pueblo, Colo.....	213	45	, June, 1921			68	R.M.
323	Arroyo Seco, Soledad, Calif.....	215	62		19		6	S.W.
324	Owens Lake Group, Calif.....	216			21	24	70	G.B.
325	Sespe Creek, Sespe, Calif.....	216	86	, Jan., 1916			66	S.W.
326	Uinta, White Rocks, Utah.....	218	10		5	20	74	R.M.
327	Logan, Logan, Utah.....	218	11		7	30	74	G.B.
328	Frazier, Granby, Colo.....	220	9		8		69	R.M.
329	Sapella, Los Alamos, N. Mex.....	221	37	, 1904			2	R.M.
330	Salmon, Altmar, N. Y.....	221	28				72	N.E.
331	San Gabriel, Azusa, Calif.....	222	212	, Feb., 1884	50	250	70	S.W.
332	Pescadero Group, San Mateo Co., Calif..	222			35	48	70	S.W.
333	Napa River Tributaries, Napa, Calif.....	226			45	61	70	S.W.
334	Ventura River, Ventura, Calif.....	226			70	87	70	S.W.
335	Smith River, Crescent City, Calif.....	227	187	, Nov., 1915			70	S.W.
336	Aicovy, near Covington, Ga.....	228	10				16	S.E.
337	Salmon Creek Group, Sonoma, Calif.....	230			25	32	70	S.W.
338	Toccoa, near Blue Ridge, Ga.....	231	53	, 1901	150	300	16	S.E.
339	Chowchilla River, Merced, Calif.....	238			35	44	70	S.W.
340	Salinas River Tributaries, Soledad, Calif..	238	80	, Jan., 1916			70	S.W.
341	Esopus Creek, Olive Bridge, N. Y.....	239	64			130	9	N.E.
342	Schoharie, Prattsville, N. Y.....	240	55			144	69	N.E.
343	Cobboscontee Stream, Gardiner, Me.....	240	14			8	15	N.W.
344	Jalama Creek, Santa Barbara, Calif.....	242				60	70	S.W.
345	West Walker River, Coleville, Calif.....	245	17	, July, 1907			70	S.W.
346	South, Port Republic, Va.....	246	37				17	N.E.
347	Mattole River, New Petrolia, Calif.....	249	223				70	S.W.
348	Patapsco, Woodstock, Md.....	251	44			28	69	N.E.
349	Crow Creek, Cheyenne, Wyo.....	251	37	, May, 1904			68	R.M.
350	Clear Creek, Redbluff, Calif.....	251			70	105	70	S.W.

† Maximum daily flow.

TABLE 2.—(Continued).

Location.*	No.	River or tributary and location.	Drainage area, in square miles.	RUN-OFF IN SECOND-FEET PER SQUARE MILE.				Main authority, see Appendix II.	Location.*
				Observed.	Expected peaks.		Rare.		
					Maximum and date.	Frequent.			
S.W.	351	Butte Creek Group, Oroville, Calif.....	251		100	134	70	S.W.	
S.W.	352	Susan, Susanville, Calif.....	256	7	5		6	S.W.	
R.M.	353	East Canada Creek, Dolgeville, N. Y.....	256	54	1913	22	37	N.E.	
R.M.	354	Cahokia Creek, Poag, Ill.....	259	14			4	N.V.	
N.E.	355	Salmon, Pulaski, N. Y.....	260	42		38	37	N.E.	
G.B.	356	Bear River, Van Trent, Calif.....	262	336	Feb., 1907	100	400	70	
S.W.	357	Mattole, Cape Mendocino, Calif.....	264			130	164	70	
S.W.	358	Tule, Portersville, Calif.....	264	25	Jan., 1914	12		70	
N.E.	359	Black, Forestport, N. Y.....	268	39				72	
S.E.	360	Whitewater, Whitewater, Calif.....	269			40	67	70	
S.W.	361	Miller's Creek, Lovella, Ore.....	270	25	1907			6	
N.E.	362	Fresno, near Fresno, Calif.....	270			36	46	70	
S.W.	363	Cottonwood Creek, Jamul, Calif.....	270	22		7		69	
S.W.	364	Nottleey, Ranger, N. C.....	272	21		15		16	
R.M.	365	Choccolocco Creek, Jenifer, Ala.....	272	43		19		69	
R.M.	366	Chewaucan, Paisley, Ore.....	272	13		6		69	
S.W.	367	Navarro River, Ukiah, Calif.....	273			54	67	70	
S.W.	368	Redwood Creek, Eureka, Calif.....	275	57				70	
S.W.	369	Goose Lake, Goose Lake, Calif.....	275			12	13	70	
N.E.	370	St. Regis, St. Regis, Mont.....	278	22	1913			72	
N.W.	371	Crooked Creek, Hileman's Farm, Pa.....	279	43				22	
S.W.	372	San Gabriel, Tributary, Azusa.....	280			65	95	70	
S.W.	373	Conejos, Mogote, Calif.....	282	15		10		69	
S.W.	374	Sutter Creek Group, Lodi, Calif.....	285			55	75	70	
S.W.	375	Piscataquis, Foxcroft, Me.....	286	78	1909	43		10	
R.M.	376	Black Fork, Hyrum, Utah.....	286	7		3		69	
S.W.	377	Yamhill, Sheridan, Ore.....	290	62		53		69	
N.E.	378	Brokenstraw Creek, Youngsville, Pa.....	290	25				22	
S.W.	379	Youghioheny, Friendsville, Md.....	294	28				22	
N.E.	380	Mora, Weber, N. Mex.....	294	94	1904			68	
N.E.	381	Panoche Creek, Mendota, Calif.....	295			30	43	70	
R.M.	382	Antietam Creek, Sharpsburg, Ind.....	295	23	1902	11		17	
S.W.	383	Carson, East Fork, Stateline, Nev.....	298	12	June, 1911			70	
G.B.	384	San Dieguito, Bernardo, Calif.....	299	241	Jan., 1916			66	
S.W.	385	Gunpowder River, Maryland.....	302	83	1889			69	
R.M.	386	Oil Creek, Rouseville, Pa.....	302	28				22	
G.B.	387	Big Thompson Creek, Loveland, Colo.....	305	7		4		69	
R.M.	388	Walker, W. Fork, Coleville, Calif.....	306	14		7		69	
R.M.	389	Honcut Creek, Oroville, Calif.....	314			90	124	70	
N.E.	390	Rapid Creek, Rapid, S. Dak.....	320	3	1904	2		72	
S.W.	391	Carson, East Fork, Nevada.....	323			14	16	70	
S.W.	392	Soquel Creek Group, Santa Cruz, Calif.....	324			50	73	70	
S.W.	393	Esopus, Kingston, N. Y.....	324	63		42		69	
S.W.	394	San Luis Rey, near Pala, Calif.....	325	231				70	
S.E.	395	San Jacinto Tributary, Calif.....	330			50	89	70	
S.W.	396	Zion Creek, near Springdale, Utah.....	330			7	20	74	
S.E.	397	Silver Creek, near Lebanon, Ill.....	335	11	Aug., 1916			4	
S.W.	398	Croton, Croton Dam, N. Y.....	339	75	1867			72	
N.E.	399	Canyon Diablo, Arch Bridge, Ariz.....	340	104	Sept., 1923			74	
S.W.	400	Carrabassett, North Anson, Me.....	340	40		27		15	
N.E.	401	Bear Creek, at Forks, Colo.....	345	7		4		69	
N.E.	402	Moose, Moose River, N. Y.....	346	20		17		69	
N.E.	403	Great Westfield, Mass.....	350	151	1878			72	
S.W.	404	Umatilla, Gibbon, Ore.....	353	28		10		69	
N.E.	405	Westfield, Massachusetts.....	356	149	1878			69	
S.W.	406	Ogden, Ogden, Utah.....	360	9		5	15	69	
N.E.	407	Battle Creek, Red Bluff, Calif.....	366			90	121	70	
R.M.	408	West Canada Creek, Hincley, N. Y.....	372	105	1869	35		37	
N.E.	409	Ocoee, McCays, Copperhill, Tenn.....	374	48		19		69	
S.W.	410	San Diego River, Santee, Calif.....	375	187	Jan., 1916			66	

† Maximum daily flow.

TABLE 2.—(Continued).

No.	River or tributary and location.	Drainage area, in square miles.	RUN-OFF, IN SECOND-FOOT PER SQUARE MILE.				Main Authority, see Appendix II.	Location.*	
			Observed.	Expected peaks.		Main Authority, see Appendix II.			Location.*
				Frequent.	Rare.				
			Maximum and date.						
411	Esopus Creek, Mt. Marion, N. Y.....	378	65		9	11	9	N.E.	
412	Surprise Valley Group, Cedarville, Calif.....	379			60	77	70	G.B.	
413	Malibu River Group, Chatsworth, Calif.....	379			35		70	S.W.	
414	Rondout Creek, Rosedale, N. Y.....	380	51				9	N.E.	
415	Pompton, Two Bridges, N. J.....	380	62	1903			1	N.E.	
416	Carson, East Fork, Gardnerville, Nev.....	381	9	1904	6		6	G.B.	
417	Saco, Center Conway, N. H.....	385	36		27		69	N.E.	
418	Roanoke, Roanoke, Va.....	388	47		23		69	S.E.	
419	Tule River, Tulare, Calif.....	390			20	28	70	S.W.	
420	Fall, Fremont, Idaho.....	390	10		8		69	R.M.	
421	Calaveras, Jenny Lind, Calif.....	394	176	Jan., 1911	170	229	70	S.W.	
422	Willow Creek Group, Colusa, Calif.....	394			60	79	70	S.W.	
423	Middle Oconee, near Athens, Ga.....	395	49	1902			16	S.E.	
424	Pacolet, Spartanburg, S. C.....	400	89	1903			19	S.E.	
425	Mojave, below Victorville, Calif.....	400	33		20	40	6	G.B.	
426	Blacklick Creek, Blacklick, Pa.....	403	38		24		69	N.E.	
427	Tygerts Valley, Belington, W. Va.....	403	41	1902			22	M.V.	
428	West Walker River, California.....	405			20	27	70	G.B.	
429	Moose, Ayers Mill, N. Y.....	407	31				72	N.E.	
430	Watauga, Elizabethton, Tenn.....	408	23		14		69	M.V.	
431	Potomac, North Branch, Piedmont, W. Va.....	410	33		20		17	N.E.	
432	Hiwassee, Murphy, N. C.....	410	54	1899	30		16	S.E.	
433	East Walker River, California.....	411			19	24	70	G.B.	
434	Mahoning Creek, Furnace Bridge, Pa.....	412	30				22	N.E.	
435	Carson, East Fork, Rodenbohs, Nev.....	414	13		9		69	G.B.	
436	Elder Creek Group, Hamilton, Calif.....	414			47	63	70	S.W.	
437	Esopus, at Saugerties, N. Y.....	417	192				69	N.E.	
438	Buffalo Creek, New York.....	420	55	1902			69	N.E.	
439	Oil Creek, near Pueblo, Colo.....	423	6	June, 1921			68	R.M.	
440	Sangamon, South Fork, Taylorsville, Iowa.....	427	10				4	M.V.	
441	Stony Creek, Johnstown, Pa.....	428	70				72	N.E.	
442	San Gabriel River, Georgetown, Tex.....	431	371	Sept., 1921			67	M.V.	
443	Youghiogheny, Confluence, Pa.....	435	55		27		69	N.E.	
444	Laramie, Woods Landing, Wyo.....	435	10		7		69	R.M.	
445	Owens River, Round Valley, Calif.....	439	3	June, 1907			70	M.V.	
446	Appalachie, Brickhead, Ga.....	440	17		11		69	S.E.	
447	Whetstone, Bigston, S. Dak.....	441	3				25	M.V.	
448	Battenkill, Greenwich, N. Y.....	444	22				37	N.E.	
449	Cow Creek, Red Bluff, Calif.....	444			90	114	70	S.W.	
450	Misquiquet, below Richford, Vt.....	445	23	1913			72	N.E.	
451	Cherry Creek, Denver, Colo.....	445	25	July, 1912			68	R.M.	
452	Bishops Creek Group, Owens Valley, Calif.....	446			25	32	70	G.B.	
453	Whiteface, below Meadowlands, Minn.....	446	13	1916			72	M.V.	
454	Truckee, Statoline, Calif.....	447	38	Mar., 1907	4		70	G.B.	
455	Casselman, Confluence, Pa.....	448	46		23		22	N.E.	
456	Tionesta Creek, Nebraska, Pa.....	451	20				22	N.E.	
457	St. Mary, near Cardston, Canada.....	452	40		13		69	C.	
458	Pine, below Pine River, Reservoir, Minn.....	452	4		3		69	M.V.	
459	Union River, near Junction, Me.....	452	50	Apr., 1923			58	N.E.	
460	Adobe Meadows Group, Owens Valley, Calif.....	453			18	21	70	G.B.	
461	Chatanika, below Poker, Alaska.....	456	8	1911			72	A.	
462	Mad River, Arcata, Calif.....	457	42	1912		142	70	S.W.	
463	Santa Ana Tributaries, Calif.....	460			50	128	70	S.W.	
464	Machias, Whitneyville, Me.....	465	24		12		69	N.E.	
465	Cattaraugus Creek, Versailles, N. Y.....	467	53				37	N.E.	
466	Lake Fork, Myton, Utah.....	475	6		4		69	R.M.	
467	Caliente Creek, near Tehachapi, Calif.....	471			30	48	70	S.W.	
468	St. Charles River, Pueblo, Colo.....	482	149	June, 1921			68	R.M.	
469	Little Wolf, Royaltown, Wis.....	485	11	1914			72	M.V.	
470	Ausable, Ausable Forks, N. Y.....	487	45	1913			37	N.E.	

† Maximum daily flow.

TABLE 2.—(Continued).

Location.*	No.	River or tributary, and location.	Drainage area, in square miles.	RUN-OFF, IN SECOND- FEET PER SQUARE MILE.				Main authority, see Appendix II.	Location.*
				Observed.		Expected peaks.			
				Maximum and date.	Frequent.	Rare.			
N. E.	471	Coquitlam, above Coquitlam Dam, British Columbia.....	490	49 ,..... 1921	25	50	94	C.	
G. B.	472	Eagle Lake Group.....	498	10	11	70	G. B.	
S. W.	473	Lake Tahoe Basin, California.....	499	20	34	70	G. B.	
N. E.	474	Yadkin, North Wilkesboro, N. C.....	500	36 ,.....	25	69	S. E.	
G. B.	475	Cache Creek, Lower Lake, Calif.....	500	8.7 ,.....	8	S. W.	
N. E.	476	Grande, North Branch, Haley, N. Dak.....	500	11.6 ,..... 1913	72	N. W.	
S. E.	477	Yakima, Cle Elum, Wash.....	500	51.2 ,..... 1915	31	72	R. M.	
S. W.	478	Shoshone, South Fork, Marquette, Wyo.....	500	10.6 ,.....	7	69	R. M.	
R. M.	479	Deerfield, Shelburne Falls, Mass.....	501	42.5 ,..... 1909	12	N. E.	
S. W.	480	Feather, North Fork, Prattville, Calif.....	506	19.5 ,.....	12	8	S. W.	
S. W.	481	Kaweah, Three Rivers, Calif.....	514	28.6 , Jan., 1916	70	S. W.	
S. E.	482	Olentangy, Columbus, Ohio.....	514	98 , Mar., 1913	28	100	71	M. V.	
S. E.	483	West Canada Creek, Middleville, N. Y.....	518	24.9 ,..... 1898	72	N. E.	
G. B.	484	Truckee, at Tahoe, Calif.....	519	2.6 ,.....	2	5	6	G. B.	
N. E.	485	Dolores, Dolores, Colo.....	524	5.6 ,.....	4	69	R. M.	
M. V.	486	Owens River (Upper) California.....	524	25	32	70	G. B.	
G. B.	487	Price, Helper, Utah.....	530	8.5 ,..... 1913	72	R. M.	
N. E.	488	Coosawattee, Carters, Ga.....	531	31.9 ,..... 1901	22	16	S. E.	
M. V.	489	Cache la Poudre, North Fork, Livermore, Colo.....	532	38.0 , May, 1904	68	R. M.	
N. E.	490	Cosumnes, Michigan Bar, Calif.....	534	42.0 , Jan., 1911	40	55	70	S. W.	
G. B.	491	Los Angeles, Tributaries, Los Angeles, Calif.....	534	71.2 , Dec., 1889	50	100	70	S. W.	
N. E.	492	Canyon Diablo, Leupp, Ariz.....	544	82 , Sept., 1923	30	90	74	R. M.	
S. W.	493	Santa Catarina, Monterey, Mexico.....	544	432± , Aug., 1909	91	M.	
N. E.	494	Ocoquan Creek, Ocoquan, Va.....	546	38.3 ,..... 1915	72	S. E.	
N. E.	495	Madeline Plains Group, N. E. California.....	548	9	11	70	G. B.	
R. M.	496	Conewanger, Frewsburg, N. Y.....	550	20.9 ,.....	22	N. E.	
M. V.	497	Uncompahgre, Montrose, Colo.....	565	3.7 ,.....	3	69	R. M.	
N. E.	498	San Luis Rey, Oceanside, Calif.....	565	169.0 , Jan., 1916	06	S. W.	
M. V.	499	Sacramento, above Pitt River, California.....	568	60	101	70	S. W.	
N. E.	500	Poso Creek Group, Tulare, Calif.....	576	20	26	70	S. W.	
R. M.	501	Stony Creek, Fruto, Calif.....	577	50.8 , Feb., 1909	28	69	S. W.	
M. V.	502	Hoosic, Buskirk, N. Y.....	579	23.6 ,.....	18	69	N. E.	
S. E.	503	Farmington, Conn.....	584	41.7 ,.....	72	N. E.	
M. V.	504	Tugaloo, near Madison, S. C.....	583	36.9 ,.....	26	68	S. E.	
N. E.	505	Sun, North Fork, Augusta, Mont.....	600	54.0 , June, 1916	74	R. M.	
S. W.	506	Fort Pierce, Wash, Utah-Arizona Boundary.....	600	6.5 , Aug., 1909	4	10	71	R. M.	
N. E.	507	Etowah, Canton, Ga.....	604	31.5 ,..... 1895	23	37	S. E.	
R. M.	508	Hoosick, Johnsonville, N. Y.....	605	38.0 ,..... 1913	69	N. E.	
G. B.	509	Pennigewasset, Plymouth, N. H.....	615	49.8 ,.....	27	59	70	N. E.	
M. V.	510	Guadalupe Group, Santa Rosa, Calif.....	623	45	204	70	S. W.	
N. E.	511	Smith River, N. W. California.....	627	160	72	N. W.	
G. B.	512	Des Plaines, Riverside, Ill.....	630	20.8 ,..... 1889	70	M. V.	
N. E.	513	Mokelumne, near Clements, Calif.....	632	32.5 , Jan., 1911	13	7	69	S. W.	
N. E.	514	Saranac, Plattsburg, N. Y.....	634	7.5 ,.....	69	N. E.	
C.	515	White, Mecker, Colo.....	634	5	70	M. V.	
M. V.	516	Alameda Creek, Sunol Glenn, Calif.....	639	23.0 , Mar., 1911	69	S. W.	
N. E.	517	Provo, Provo, Utah.....	640	6.4 ,.....	69	G. B.	
G. B.	518	Sandy, Madison, Me.....	650	21.2 ,.....	15	69	N. E.	
A.	519	Pigeon, Newport, Tenn.....	655	30.9 ,.....	15	70	M. V.	
S. W.	520	Putah Creek, Winters, Calif.....	655	91.6 , Dec., 1913	23	17	S. W.	
N. E.	521	Monocacy, near Frederick, Md.....	660	31.0 ,..... 1902	22	73	N. E.	
N. E.	522	Grande Ronde, Hilgard, Ore.....	660	7.0 ,.....	5	69	N. W.	
R. M.	523	Milwaukee, Milwaukee, Wis.....	661	8.0 ,..... 1915	73	M. V.	
S. W.	524	Russian, Geyersville, Calif.....	662	24.9 ,.....	70	S. W.	
R. M.	525	Tuckasegee, Bryson, N. C.....	662	58.2 ,..... 1889	34	73	S. E.	
M. V.	526	McCloud, Baird, Calif.....	669	82.2 , Feb., 1904	33	69	S. W.	
N. E.	527	Spanish Fork, Spanish Fork, Utah.....	670	2.9 ,.....	2	69	G. B.	
S. W.	528	Uinta, Fort Duchesne, Utah.....	672	6.7 ,.....	4	69	R. M.	
R. M.	529	Black, Neillville, Wis.....	675	34.2 ,.....	15	69	M. V.	
N. E.	530	Little Tennessee, Jackson, N. C.....	675	85.3 ,..... 1901	40	21	S. E.	

† Maximum daily flow.

TABLE 2.—(Continued).

No.	River or tributary, and location.	Drainage area, in square miles.	RUN-OFF, IN SECOND-FEET, PER SQUARE MILE.				Main authority, see Appendix II.	Location.*
			Observed.	Expected peaks.				
				Maximum and date.	Frequent.	Rare.		
531	Delaware, West Branch, Hancock, N. Y.	680	49.6	28		69	N.E.	
532	Santa Margarita, near San Diego, Calif.	690		40	67	70	S.W.	
533	Willamette, Coast Fork, Goshen, Ore.	690	45.4	30		69	N.W.	
534	Blue, Kremmling, Colo.	700	8.6	8		69	R.M.	
535	Moye, Snider, Idaho.	717	11.1		1913	72	S.W.	
536	San Jacinto, Elsmore, Calif.	718	19.5	Jan.	1916	66	S.W.	
537	Trinity, Lewiston, Calif.	718	37.4	Jan.	1914	70	N.W.	
538	Racquette, Piercefield, N. Y.	723	8.1			5	N.E.	
539	Salmon, Somes Bay, Calif.	734	44.9	Feb.	1912	70	N.W.	
540	Big Muddy, near Cambon, Ill.	735	15.0			4	M.V.	
541	Indian Creek, Crescent Mills, Calif.	740	15.4		9	69	S.W.	
542	Purgatoire, Trinidad, Colo.	742	61.0	Sept.	1904	68	R.M.	
543	West Fork, Enterprise, W. Va.	744	23.7		1911	22	M.V.	
544	Appomatox, Mottoax, Va.	745	15.7		11	69	S.E.	
545	Santa Ynez, near Lompoc, Calif.	750	55.3	Feb.	1915	70	S.W.	
546	Chico Creek, near Pueblo, Colo.	750	38.0	June,	1920	68	R.M.	
547	Huron, Geddes, Mich.	757	4.9		3	69	M.V.	
548	Catawba, Morganton, N. C.	758	42.4		28	69	S.E.	
549	Broad, near Carlton, Ga.	762	38.2		1902	27	16	
550	Passaic, Little Falls, N. J.	773	24.2		1882		72	
551	Chagres, Bohio, Panama.	779	115.5				32	
552	Noyo River Group, N. W. California.	780			60	78	70	
553	Youghiogheny, Pennsylvania.	782	58.9		1888		69	
554	Little Muddy, Williston, N. Dak.	800	5.4		2	69	N.E.	
555	Pryor Creek, Huntley, Mont.	800	1.7		1	69	M.V.	
556	Escanaba, Escanaba, Mich.	800	13.4		9	69	R.M.	
557	Eagle, Gypsum, Colo.	800	7.5		6	69	M.V.	
558	Shasta River, California.	803			12	13	70	
559	Hudson, North Creek, N. Y.	804	35.1				37	
560	North Point, Republic, Va.	804	29.7		1896		17	
561	Raritan, Bound Brook, N. J.	806	64.5		1882	25	1	
562	Animas, Durango, Calif.	812	9.6			6	69	
563	Scott River, Scott Bar, Calif.	813			30	39	70	
564	Snake, South Fork, Moran, Wyo.	820	25.5		11	69	R.M.	
565	Passaic, Dundee, N. J.	823	43.4		1903	15	71	
566	Kettle, near Sandstone, Minn.	825	7.1		1912		25	
567	Holston, South Fork, Bluff City, Tenn.	828	39.8		1902	24	72	
568	James, North Fork, Glasgow, Va.	831	44.8		1896	21	71	
569	West Gallatin, Salesville, Mont.	860	12.5			7	69	
570	Silvie, Burns, Ore.	865	5.5			3	69	
571	Dead, near The Forks, Me.	870	20.7				15	
572	Youghiogheny, below Confluence, Pa.	874	52.6			1	71	
573	Deschutes, Allen Ranch, Lava, Ore.	880	2.4			1	69	
574	San Pitch, Gunnison, Utah.	886	10.6	Mar.,	1915	0.6	87	
575	Fish, Wallagrass, Me.	890	10.0			8	69	
576	Potomac North Branch, Cumberland, Md.	891	22.8		1897		72	
577	Flint, Molena, Ga.	892	7.4				71	
578	Black, Lyons Falls, N. Y.	897	46.0		1869		72	
579	Manistee, Sherman, Mich.	900	3.2			3	69	
580	Two Butte Creek, at Mouth, Colo.	900	39.0	Oct.,	1908		68	
581	Schoharie Creek, Fort Hunter, N. Y.	900	55.1	Oct.	1901		37	
582	Tule Lake Group, near Mt. Shasta, Calif.	901				14	16	
583	Clarion, Clarion, Pa.	910				26		
584	Santa Clara Tributaries, Ventura, Calif.	911	43.2			60	82	
585	Delaware, East Branch, Hancock, N. Y.	920	78.8		1904	36		
586	Fountain Creek, Pueblo, Colo.	932	54.0	June,	1921			
587	Cottonwood Creek, near Red Bluff, Calif.	937				60	75	
588	Clearwater, South Fork, Grangeville, Idaho	940	10.5		1912			
589	Chelan, Chelan, Wash.	950	10.3			9	54	
590	Truckee, near Stateline, Calif.	955	16.0			6	6	

† Maximum daily flow.

TABLE 2.—(Continued).

No.	River or tributary, and location.	Drainage area, in square miles.	RUN-OFF, IN SECOND-FEET PER SQUARE MILE.				Main authority, see Appendix II.	Location.*	
			Observed.	Expected peaks.		Frequent.			Rare.
				Maximum and date.					
591	Mahoning, Youngstown, Ohio.....	958	20.2	0.15	69	M.V.	
592	Teton, near St. Anthony, Idaho.....	960	7.9	5	69	R.M.	
593	Mill Creek Group, near Red Bluff, Calif..	971	90	122	70	S.W.	
594	Cut Bank Creek, Cut Bank, Mont.....	971	9.1 1908	72	R.M.	
595	Stanislaus, Knight's Ferry, Calif.....	983	61.0	Jan., 1911	27	70	S.W.	
596	Winooski, Richmond, Vt.....	985	29.8 1904	20	72	N.E.	
597	French Broad, Asheville, N. C.....	987	31.2	16	69	S.E.	
598	Flint, near Woodbury, Ga.....	988	30.6 1913	16	16	S.E.	
599	Huron, Flat Rock, Mich.....	1 000	2.8	2	69	M.V.	
600	Redwater, Belle Fourche, S. Dak.....	1 006	8.0 1904	72	M.V.	
601	Virgin, Virgin City, Utah.....	1 010	11.9 1912	6	13	72	R.M.	
602	San Luis Obispo Group, Monterey, Calif..	1 019	55	73	70	S.W.	
603	Hoosatic, Gaylordsville, Conn.....	1 020	31.0	16	69	N.E.	
604	Blackfoot, Preston, Idaho.....	1 020	2.3	2	69	G.B.	
605	Shenandoah, North Fork, Riverton, Va...	1 037	20.9 1901	17	S.E.	
606	Cahaba, Centerville, Ala.....	1 040	16.4	11	69	S.E.	
607	Snake, North Fork, Ora, Idaho.....	1 040	5.2	4	69	R.M.	
608	Scioto, Columbus, Ohio.....	1 047	80.8	Mar., 1913	14	39	M.V.	
609	Merced, near Merced Falls, Calif.....	1 054	36.0	Jan., 1911	14	70	S.W.	
610	Saluda, Waterloo, S. C.....	1 056	18.0 1903	72	S.E.	
611	Sacadaga, Hadley, N. Y.....	1 060	27.4	37	N.E.	
612	Cache la Poudre, Fort Collins, Colo.....	1 060	5.3	3	69	R.M.	
613	Genesee, Mount Morris, N. Y.....	1 070	39.2	72	N.E.	
614	Truckee, Reno, Nev.....	1 070	7.0 1913	72	G.B.	
615	French Creek, Carlton, Pa.....	1 070	22.9	22	N.E.	
616	Pajaro Tributaries, near Salinas, Calif.....	1 070	44	62	70	S.W.	
617	Penobscot, East Branch, Grindstone, Me...	1 070	28.9	Apr., 1922	14	88	N.E.	
618	Oconee, Greensboro, Ga.....	1 100	62.0	16	69	S.E.	
619	Walker, Yerington, Nev.....	1 100	1.5	1	69	G.B.	
620	Nollchucky, Greeneville, Tenn.....	1 100	29.4	14	69	M.V.	
621	Naches, North Yakima, Wash.....	1 120	19.6	10	69	N.W.	
622	Cowlitz, Mossy Rock, Wash.....	1 170	43.5 1906	72	N.W.	
623	Roquette, Massena Springs, N. Y.....	1 170	9.4	7	5	N.E.	
624	Chattahoochee, Norcross, Ga.....	1 170	25.8	15	69	S.E.	
625	Neuse, Selma, N. C.....	1 175	6.7	70	S.E.	
626	Pea, Pera, Ala.....	1 180	10.7	8	69	S.E.	
627	Hiwassee, Reliance, Tenn.....	1 180	46.7	24	69	M.V.	
628	Cache Creek, Yolo, Calif.....	1 195	17.4	Feb., 1909	12	70	S.W.	
629	Umatilla, Yoakum, Ore.....	1 200	20.0	8	69	N.W.	
630	Yuba, near Smartsville, Calif.....	1 200	92.5	Jan., 1909	47	70	S.W.	
631	Wenatchee, Dryden, Wash.....	1 200	20.1 1913	14	72	N.W.	
632	Deschutes West Ranch, Lava, Ore.....	1 240	3.2	2	69	N.W.	
633	Heart, Richardson, N. Dak.....	1 250	6.4	8	72	M.V.	
634	Thunder Bay, Alpena, Mich.....	1 260	5.8	4	69	M.V.	
635	Dock, Columbia, Tenn.....	1 260	20.3	15	69	M.V.	
636	Conecuts, Beck, Ala.....	1 290	12.7	8	69	S.E.	
637	Animas, Artec, N. Mex.....	1 300	9.6	5	69	R.M.	
638	Mohawk, Little Falls, N. Y.....	1 306	26.6 1913	15	37	N.E.	
639	Oneida, Euclid, N. Y.....	1 310	19.5	8	69	N.E.	
640	Wapsipinicon, Stone City, Iowa.....	1 310	6.6	5	69	M.V.	
641	Chagres, near Gatun, Panama.....	1 320	93.9	32	P.	
642	Youghiogheny, Connellsville, Pa.....	1 320	41.2 1907	72	N.E.	
643	Tygart Valley, Feltsman, W. Va.....	1 327	26.4	22	M.V.	
644	Orestimba Creek Group, Merced, Calif..	1 340	30	40	70	S.W.	
645	Tejon Creek Group, near Hanford, Calif..	1 311	21	35	70	S.W.	
646	Greenbriar, Alderson, W. Va.....	1 344	46.5	27	69	M.V.	
647	Grande Ronde, Elgin, Ore.....	1 350	6.2	4	69	N.W.	
648	Cheat, Morgantown, W. Va.....	1 380	30.3	30	22	M.V.	
649	Hondo Reservoir, N. Mex.....	1 387	4.6 1904	72	R.M.	
650	San Juan, Arboles, Colo.....	1 390	28.8 1911	72	R.M.	

† Maximum daily flow.

TABLE 2.—(Continued).

No.	River or tributary and location.	Drainage area, in square miles.	RUN-OFF IN SECOND-FOOT PER SQUARE MILE.				Main authority see Appendix II.	Location*.	
			Observed.	Expected peaks.		Frequent.			Rare.
				Maximum and date.					
651	Ocmulgee, Jackson, Ga.....	1 400	18.2	13	69	S.E.	
652	Rio Grande, Del Norte, Calif.....	1 400	5.5	3	69	S.W.	
653	Genesee, New York.....	1 410	12.5	7	37	N.E.	
654	St. Croix, Woodland, Me.....	1 420	14.3	2	69	N.E.	
655	Au Sable, Bamfield, Mich.....	1 420	3.0	7	69	M.V.	
656	Potomac, Springfield, W. Va.....	1 440	17.8	17	N.E.	
657	Willamette, Middle Fork, Jasper, Ore.....	1 450	64.2	33	69	N.W.	
658	Merrimac, Franklin Jct., N. H.....	1 460	19.1	13	69	N.E.	
659	Kalamazoo, Allegan, Mich.....	1 470	7.0	3	69	M.V.	
660	Shoshone, Cody, Wyo.....	1 480	10.7	8	69	R.M.	
661	Canadian, French, N. Mex.....	1 480	105.0	Oct., 1909	68	M.V.	
662	Mattawamkeag, Mattawamkeag, Me.....	1 500	16.3	11	69	N.E.	
663	Honey Lake Group, California.....	1 507	11	14	70	G.B.	
664	Russian, N. W. California.....	1 508	40	51	70	N.W.	
665	Truckee, Yista, Nev.....	1 520	5.9	3	69	G.B.	
666	Oostanaula, Resaca, Ga.....	1 527	14.5	71	S.E.	
667	Chenango, Binghamton, N. Y.....	1 530	23.5	18	69	N.E.	
668	Catawba, Catawba, N. C.....	1 535	61.9	May, 1901	19	34	S.E.	
669	Yakima, Umtanum, Wash.....	1 540	26.7	16	69	N.W.	
670	Tuolumne, La Grange, Calif.....	1 543	39.0	Jan., 1911	6	70	S.W.	
671	Bitterroot, Grantsdale, Mont.....	1 550	8.3	6	69	R.M.	
672	Chattahoochee, Oakdale, Ga.....	1 560	31.3	22	16	S.E.	
673	Kennebec, Forks, Me.....	1 570	13.2	Apr., 1923	9	88	N.E.	
674	Lower Scioto, Columbus, Ohio.....	1 570	70.5	1913	20	17	M.V.	
675	Shenandoah, South Fork, Front Royal, Va.....	1 570	48.9	1902	16	71	S.E.	
676	Osgewatchie, Ogdensburg, N. Y.....	1 580	10.0	8	69	N.E.	
677	Oostanaula, Branch, Mobile, Ga.....	1 610	24.3	15	69	S.E.	
678	San Joaquin, Friant, Calif.....	1 631	36.6	1880	11	70	S.W.	
679	Santa Maria, Santa Maria, Calif.....	1 634	40	61	70	S.W.	
680	Allegheny, Red House, N. Y.....	1 610	25.0	15	22	N.E.	
681	Weiser, Weiser, Idaho.....	1 670	10.7	6	69	N.W.	
682	Kings, near Sanger, Calif.....	1 694	35.3	Jan., 1914	12	70	S.W.	
683	Elk, Elkmont, Ala.....	1 700	28.8	24	69	S.E.	
684	Methow, Pateros, Wash.....	1 710	7.0	7	69	N.W.	
685	Yampa, Craig, Colo.....	1 730	5.6	5	69	R.M.	
686	Arkansas Tributaries, Pueblo, Colo.....	1 740	57.0	June, 1921	91	R.M.	
687	Truckee, Clark, Nev.....	1 740	3.0	72	G.B.	
688	King River, State Point, Calif.....	1 742	25.2	1901	72	S.W.	
689	Kiskiminitis, Avonmore, Pa.....	1 750	44.3	26	69	N.E.	
690	Umpqua, South Fork, Brockway, Ore.....	1 800	39.2	23	69	N.W.	
691	Etowah, Rome, Ga.....	1 800	33.0	195	69	S.E.	
692	Gallatin, Logan, Mont.....	1 805	3.6	3	69	R.M.	
693	Black, Carthage, N. Y.....	1 812	21.2	10	72	N.E.	
694	Penobscot, West Branch Millinocket, Me.....	1 880	12.9	8	10	N.E.	
695	Black Warrior, Cordova, Ala.....	1 900	28.8	21	69	S.E.	
696	American, near Folsom, Calif.....	1 900	99.5	Jan., 1862	70	S.W.	
697	Schuylkill, Fairmont, Pa.....	1 920	42.8	16	69	N.E.	
698	Feather, North Fork, Big Bend, Calif.....	1 940	56.3	8	8	S.W.	
699	Rogue River, Tolo, Ore.....	2 020	23.9	1909	17	72	N.W.	
700	Chemung, Elmira, N. Y.....	2 055	67.1	1889	72	N.W.	
701	Missouri, Red Bluffs, Mont.....	2 085	5.0	3	69	R.M.	
702	Flambeau, Ladysmith, Wis.....	2 120	6.0	5	69	M.V.	
703	Meadow Valley Wash, near Moapa, Nev.....	2 150	3.8	Jan., 1910	74	S.W.	
704	Birch Creek, Fourteen Mile House, Alaska.....	2 150	6.9	1911	72	A.	
705	Palouse, Hooper, Wash.....	2 210	7.8	3	69	N.W.	
706	San Francisco Bay, Drainage Basins, Calif.....	2 219	40	51	70	S.W.	
707	Aroostook, Fort Fairfield, Me.....	2 230	15.4	14	69	N.E.	
708	Payette, Horse Shoe Bend, Idaho.....	2 240	8.1	6	69	N.W.	
709	Tar, Tarboro, N. C.....	2 290	6.4	71	S.E.	
710	Hiwassee, Charleston, Tenn.....	2 297	20.0	16	69	M.V.	

† Maximum daily flow.

TABLE 2.—(Continued).

Location.*	No.	River or tributary, and location.	Drainage area, in square miles.	RUN-OFF, IN SECOND-FEET PER SQUARE MILE.				Main authority, see Appendix II.	Location.*
				Observed.	Expected peaks.				
					Maximum and date.	Frequent.	Rare.		
S. E.	711	Trinity, Fort Worth, Tex.	2 300	20.0	Apr., 1922	13		74	M.V.
S. W.	†712	Androscooggin, Rumford Falls, Me.	2 320	23.8	1869	11		15	N.E.
N. E.	713	Klamath, Northwestern California.	2 320			15	20	70	N.W.
N. E.	714	Merrimac, Garvins Falls, N. H.	2 340	18.3		14		69	N.E.
M. V.	715	Susquehanna, Binghamton, N. Y.	2 350	26.6	1902	16		72	N.E.
N. E.	716	Muskegon, Newdygo, Mich.	2 350	2.7		2		69	M.V.
N. W.	†717	Genesee, Rochester, N. Y.	2 365	21.2		10		71	N.E.
N. E.	718	Grand, Kremmling, Colo.	2 380	6.4		4		69	R.M.
M. V.	719	Guadalupe, New Braunfels, Tex.	2 400	23.6	Sept., 1921	4		67	M.V.
M. V.	720	Kern, Bakersfield, Calif.	2 410	7.6	Jan., 1914	2		70	S.W.
N. E.	721	Menominee, near Iron Mountain, Mich.	2 415	6.2		5		69	M.V.
G. B.	722	Ocmulgee, Macon, Ga.	2 425	21.0	1902	14		16	S.E.
N. W.	723	Miami, Dayton, Ohio.	2 450	106.	Mar., 1913	19		71	M.V.
G. B.	724	Little Tennessee, McGhee, Tenn.	2 470	28.3		15		69	M.V.
S. E.	725	Elkhorn, near Norfolk, Nebr.	2 470	3.2	1903			14	M.V.
N. E.	726	Tallapoosa, Sturdevant, Ala.	2 500	23.6		15		69	S.E.
S. E.	727	Clark Fork, Fromberg, Mont.	2 500	4.3		3		69	R.M.
N. E.	728	Sangamon, Riverton, Ill.	2 560	7.5	1911			4	M.V.
N. W.	729	Sevier, Marysville, Utah.	2 560	1.2		1		69	G.B.
S. W.	730	Rhine, Macon, Ga.	2 574	37.4				69	S.E.
R. M.	731	Grand River, Ontario, Canada.	2 600	14.3				79	C.
S. E.	732	Marias, Shelby, Mont.	2 610	11.3		5		69	R.M.
N. E.	733	Yellowstone, Corwin Springs, Mont.	2 630	8.7	1911			72	R.M.
M. V.	734	Wisconsin, near Merrill, Wis.	2 630	8.0		6		18	M.V.
S. E.	735	Pecos, Santa Rosa, N. Mex.	2 649	17.6	1904			72	M.V.
N. E.	736	James, Buchanan, Va.	2 660	23.3		16		69	S.E.
S. W.	737	Kennebec, near Waterville, Me.	2 700	48.6	1901			15	N.E.
N. E.	738	Flint, Montezuma, Ga.	2 700	7.7		5		69	S.E.
S. W.	739	Savannah, near Calhoun Falls, S. C.	2 712	27.8				16	S.E.
N. W.	740	New, Radford, Va.	2 725	63.8	1900	30		21	S.E.
S. E.	741	Saline, Beverly, Kans.	2 730	5.9	1896			19	M.V.
N. W.	742	Duchesne, Myton, Utah.	2 750	3.5		2		69	R.M.
S. E.	743	Dan, South Boston, Va.	2 750	16.1		8		69	S.E.
N. W.	744	Hudson, Glens Falls, N. Y.	2 760	25.4	1913			37	N.E.
R. M.	745	Canadian, Taylor, N. Mex.	2 832	32.1	1904			72	M.V.
R. M.	746	Oconee, Milledgeville, Ga.	2 840	10.4		6		69	S.E.
G. B.	747	Humboldt, Elko, Nev.	2 840	0.8		0.5		69	G.B.
S. W.	748	Trinity, near Dunsmuir, Calif.	2 965			35	44	70	S.W.
N. E.	749	Catawba, near Rockhill, S. C.	2 987	50.5	May, 1901	22		34	S.E.
N. W.	750	Shenandoah, Millville, W. Va.	2 995	46.6	1896	15		17	S.E.
S. E.	751	Salt Creek, at Mouth, N. M.	3 052	4.1	1904			72	M.V.
R. M.	752	Holston, Rogersville, Tenn.	3 060	16.8		12		69	M.V.
N. E.	753	Arkansas, Canon City, Colo.	3 060	2.2		2		69	R.M.
N. E.	754	Verdigris, Liberty, Kans.	3 067	16.4	July, 1904	13		2	M.V.
S. E.	755	Eel, Scotia, Calif.	3 071	94.4				70	S.W.
S. W.	756	Seneca, Baldwinsville, N. Y.	3 100	3.5		3		69	N.E.
N. E.	757	Connecticut, Fairlee, Vt.	3 100	18.5	1913			72	N.E.
S. W.	758	Link, Klamath Falls, Ore.	3 110	2.9				6	N.W.
N. W.	759	Pearl, Jackson, Miss.	3 120	11.7		7		69	S.E.
N. W.	760	Klamath, Keno, Ore.	3 150	2.7				6	N.W.
R. M.	761	Wabash, Logansport, Ind.	3 163	18.0				2	M.V.
M. V.	762	Delaware, Port Jervis, N. Y.	3 250	33.2		18		69	N.E.
S. W.	763	Belle Fourche, Belle Fourche, S. Dak.	3 250	18.2		12		69	N.E.
A.	764	Bitterroot, Missoula, Mont.	3 260	11.5		7		69	R.M.
N. W.	765	Yakima, Union Gap, near Yakima, Wash.	3 300	19.3	1904	9		69	N.W.
S. W.	766	Chattahoochee, West Point, Ga.	3 300	26.9	1901	14		16	S.E.
N. E.	767	Connecticut, Orford, N.H.	3 305	15.0		9		69	N.E.
N. W.	768	Iowa, Iowa City, Iowa.	3 320	5.9		3		69	M.V.
S. E.	769	Juniata, Newport, Pa.	3 380	53.8	1889	18		72	N.E.
M. V.	770	Mohawk, Rexford, N. Y.	3 384	23.1	1892			72	N.E.

† Maximum daily flow.

TABLE 2.—(Continued).

No.	River or tributary, and location.	Drainage area, in square miles.	RUN-OFF, IN SECOND-FOOT PER SQUARE MILE.				Main authority, see Appendix II.	Location.*
			Observed.	Expected peaks.				
				Maximum and date.	Frequent.	Rare.		
771	Saline, Saline, Kans.....	3 311	2.4	1	69	M.V.
772	Yadkin, Salisbury, N. C.....	3 399	38.3 1889	19	72	S.E.
773	Mohawk, Cohoes, N. Y.....	3 472	28.5 1913	14	37	N.E.
774	Staunton, Clarksville, Va.....	3 546	10.3	71	S.E.
775	Eel, California.....	3 547	90	121	70	S.W.
776	Crow Wing, near Mouth, Minn.....	3 580	2.8	25	M.V.
777	Yellowstone, Livingstone, Mont.....	3 580	7.5	5	69	R.M.
778	Feather, Oroville, Calif.....	3 627	51.6	Mar., 1907	26	70	S.W.
779	Cannon Ball, Stevenson, N. Dak.....	3 650	1.6	1	69	M.V.
780	Neosho, Iola, Kans.....	3 670	20.3	July, 1904	14	2	M.V.
781	Dan, Clarksville, Va.....	3 749	8.8	71	S.E.
782	South Platte, South Fork, Denver, Colo.....	3 840	1.4	0.5	69	R.M.
783	Tallapoosa, Milledge, Va.....	3 840	18.2 1901	72	S.E.
784	St. Joseph, Buchanan, Mich.....	3 940	4.7	3	69	M.V.
785	Sevier, Gunnison, Utah.....	3 990	0.6	0.3	69	G.B.
786	Spokane River, Spokane, Wash.....	4 000	8.8 1894	6	72	N.W.
787	Coosa, Rome, Ga.....	4 006	16.0 1901	14	16	S.E.
788	Pit, Bieber, Calif.....	4 040	6.8	3	8	S.W.
789	Salinas Tributaries, Salinas, Calif.....	4 042	60	112	70	S.W.
790	Merrimac, Lowell, Mass.....	4 085	19.8	72	M.V.
791	White, near Interior, S. Dak.....	4 090	4.0 1905	72	M.V.
792	Oconee, Dublin, Ga.....	4 182	8.3	6	16	S.E.
793	Malheur, Vale, Ore.....	4 190	3.5	2	69	N.W.
794	Kennebec, Waterville, Me.....	4 270	35.4 1901	15	N.E.
795	Tombigbee, Columbus, Miss.....	4 440	11.4	8	69	S.E.
796	Cape Fear, Fayetteville, N. C.....	4 493	20.2	12	69	S.E.
797	Waterce, Camdem, S. C.....	4 500	8.1	7	69	S.E.
798	Hudson, Mechanicsville, N. Y.....	4 500	26.7 1913	11	37	N.E.
799	Susquehanna, Williamsport, Pa.....	4 500	11.6	71	N.E.
800	Mississippi, above Sandy River, Minn.....	4 510	2.1	1.4	25	M.V.
801	Colorado (Grand), Glenwood Springs, Colo.....	4 520	6.1	4.5	69	R.M.
802	Merrimac, Lawrence, Mass.....	4 553	23.4	12	72	N.E.
803	Arkansas, Pueblo, Colo., before 1914.....	4 600	2.4	1.2	69	R.M.
804	Arkansas, Pueblo, Colo., after 1921.....	4 600	22.3	June, 1921	3	68	R.M.
805	Broad, Alston, S. C.....	4 609	28.4 1901	17	72	S.E.
806	Yadkin, Norwood, N. C.....	4 614	13.7	71	S.E.
807	Potomac, Dam No. 5, Md.....	4 640	22.2	72	N.E.
808	Clearwater, Kamiah, Idaho.....	4 850	15.8 1913	72	N.W.
809	Willamette, Albany, Ore.....	4 860	62.2 1861	25	72	N.W.
810	Grand, Grand Rapids, Mich.....	4 900	10 1906	72	M.V.
811	Black Warrior, Tuscaloosa, Ala.....	4 900	38.8 1895	28	72	S.E.
812	Flint, Albany, Ga.....	5 000	8.5	6	69	S.E.
813	Guadalupe, near Cuero, Tex.....	5 020	14.2 1903	72	M.V.
814	John, Fort Kent, Me.....	5 280	14.3	12	69	N.E.
815	Little River, Leon Junction, Tex.....	5 300	62.5	Sept., 1921	67	M.V.
816	Chippewa, Chippewa Falls, Wis.....	5 300	12.1	7	69	M.V.
817	Red Lake, Crookston, Minn.....	5 320	2.7	1.6	25	M.V.
818	Pit, Ydalpom, Calif.....	5 346	8.8	Dec., 1913	5	70	S.W.
819	Monongahela, Lock No. 4, Pa.....	5 430	38.1 1888	1	N.E.
820	Snake, South Fork, Lyon, Idaho.....	5 480	9.4	6	69	N.W.
821	Yakima, Kiona, Wash.....	5 520	11.5 1906	72	N.W.
822	Sevier, Leamington, Utah.....	5 595	0.4	0.3	69	G.B.
823	Susquehanna, Williamsport, Pa.....	5 640	29.0	19	69	N.E.
824	Salt River, Roosevelt, Ariz.....	5 756	36.0 1893	72	R.M.
825	Little Missouri, Medora, N. Dak.....	5 780	3.3	2	69	M.V.
826	St. Croix, St. Croix Falls, Minn.....	5 950	6.0	72	M.V.
827	Missoula, Missoula, Mont.....	5 960	6.0	3.5	69	R.M.
828	Elkhorn, Arlington, Nebr.....	5 980	1.6	1	69	M.V.
829	Bear, Collinston, Utah.....	6 000	1.9	1	69	G.B.
830	Verde, McDowell, Ariz.....	6 000	24.0 1893	72	R.M.

† Maximum daily flow.

TABLE 2.—(Continued).

No.	River or tributary, and location.	Drainage area, in square miles.	RUN-OFF, IN SECOND-FOOT PER SQUARE MILE.				Main authority, see Appendix II.	Location.*	
			Observed.	Expected peaks.		Frequent.			Rare.
				Maximum and date.					
831	Niobara, near Valentine, Nebr.....	6 070	1.1	0.6	14	M.V.	
832	Pecos, Fort Sumner, N. Mex.....	6 191	7.3 1904	72	M.V.	
833	Fox, Rapid Croche Dam, Wis.....	6 200	2.5 1895	18	M.V.	
834	New, Fayette, W. Va.....	6 200	17.8 1899	13	72	M.V.	
835	James, Cartersville, Va.....	6 230	13.6	9	69	S.E.	
836	Salt, McDowell, Ariz.....	6 260	22.0	7	69	R.M.	
837	Rock, below Rockton, Ill.....	6 290	4.3	4	M.V.	
838	Cedar, Cedar Rapids, Iowa.....	6 320	8.3	5	69	M.V.	
839	Delaware, Riegelsville, N. J.....	6 430	27.4	15	69	N.E.	
840	Penobscot, West Enfield, Me.....	6 600	14.1	9	69	N.E.	
841	Chippewa, Eau Claire, Wis.....	6 740	9.0 1905	6	1	M.V.	
842	Susquehanna, Northumberland, Pa.....	6 800	17.5 1889	71	N.E.	
843	Solomon, Miles, Kans.....	6 815	1.6	0.8	69	M.V.	
844	Coosa, Riverside, Ala.....	6 850	10.5 1898	72	S.E.	
845	Delaware, Stockton, N. J.....	6 855	37.2 1841	69	N.E.	
846	Little River, Cameron, Tex.....	7 010	92.3	Sept., 1921	67	M.V.	
847	Coosa, (Branch Mobile), Riverside, Ala.....	7 060	10.7	8	69	S.E.	
848	Mississipp.....	7 283	1.49	71	M.V.	
849	Cheyenne, Edgemont, S. Dak.....	7 350	1.5	1	69	M.V.	
850	Green River, Green River Wyo.....	7 450	3.0	2	69	R.M.	
851	Savannah, Augusta, Ga.....	7 500	40.0 1884	15	1	S.E.	
852	Penobscot, Bangor, Me.....	7 700	14.94	10	N.E.	
853	Connecticut, Sunderland, Mass.....	7 700	13.4	10	69	N.E.	
854	John Day, McDonald, Ore.....	7 800	2.9	2	69	N.W.	
855	Gunnison, Whitewater, Colo.....	7 863	3.6 1905	7	1	R.M.	
856	Smoky Hill, Ellsworth, Kans.....	7 960	2.6 1895	19	M.V.	
857	Wisconsin, Kilburn, Wis.....	8 000	10.0	90	M.V.	
858	Bighorn, Thermopolis, Wyo.....	8 184	2.1	1.6	69	R.M.	
859	St. John, Van Buren, Me.....	8 270	16.2	Apr., 1923	88	N.E.	
860	Roanoke, Old Gaston, N. C.....	8 350	32.90 1877	72	S.E.	
861	Colorado, (Grand), Palisade, Colo.....	8 546	4.9	1	R.M.	
862	Connecticut, Holyoke, Mass.....	8 660	21.10 1884	71	N.E.	
863	Allegheny, Kittanning, Pa.....	8 690	26.7	14	69	N.E.	
864	Tombigbee, Epes, Ala.....	8 830	6.9	5	69	S.E.	
865	Great Basin Drainage, Calif.....	8 876	20	25	70	G.B.	
866	Kanawha, Charleston, W. Va.....	9 900	13.49 1875	71	M.V.	
867	Jefferson, Sappington, Mont.....	8 984	1.7	1.1	69	R.M.	
868	Tennessee, Knoxville, Tenn.....	8 990	17.5	10	69	M.V.	
869	Allegheny, Kittanning, Pa.....	9 010	26.66	22	N.E.	
870	Deschutes, Biggs, Ore.....	9 180	3.8	2	69	N.W.	
871	Sacramento, Red Bluff, Calif.....	9 258	30.0 1909	15	70	S.W.	
872	Blue, near Manhattan, Kans.....	9 490	9.13 1903	72	M.V.	
873	Potomac, Point of Rocks, Md.....	9 654	48.9 1889	12	17	N.E.	
874	Susquehanna, Wilkes-Barre, Pa.....	9 810	22.2	13	1	N.E.	
875	Connecticut, Hartford, Conn.....	10 234	20.3 1854	11	11	N.E.	
876	Susquehanna, Danville, Pa.....	11 100	27.4 1902	13	69	N.E.	
877	Rio Grande, Rio Grande, N. Mex.....	11 250	2.75 1904	72	R.M.	
878	Potomac, Great Falls, Md.....	11 427	41.15 1889	72	N.E.	
879	Canadian River, Logan, N. Mex.....	11 440	11.29 1904	72	R.M.	
880	North Platte, Pathfinder, Wyo.....	12 000	1.0	0.8	69	R.M.	
881	Yellowstone, Huntley, Mont.....	12 000	4.03 1907	72	R.M.	
882	Salt River, Arizona.....	12 000	24.7 1891	72	R.M.	
883	Mississippi, Sank Rapids, Mich.....	12 400	4.1	3	69	M.V.	
884	Illinois, Peoria, Ill.....	13 480	5.94 1904	72	M.V.	
885	Loupe, Columbus, Nebr.....	13 540	5.17 1896	3	1	M.V.	
886	South Pacific Drainage, California.....	13 589	50	86	70	S.W.	
887	Salmon, Whitebird, Idaho.....	13 600	5.97 1913	72	R.M.	
888	Humboldt, Oreana, Nev.....	13 800	0.22 1897	72	G.B.	
889	Yazoo, Miss.....	13 850	10.04	72	M.V.	
890	Des Moines, Keosauqua, Iowa.....	14 300	7.0	4	69	M.V.	

† Maximum daily flow.

TABLE 2.—(Continued).

No.	River or tributary, and location.	Drainage area, in square miles.	RUN-OFF, IN SECOND FEET PER SQUARE MILE.				Main authority, see Appendix II.	Location.*
			Observed.	Expected peaks.				
				Maximum and date.	Frequent.	Rare.		
891	Missouri, Townsend, Mont.	14 500	8.6	2		69	R.M.	
892	Minnesota, Maukato, Minn.	14 600	3.0	1.6		25	M.V.	
893	North Platte, Orin Junction, Wyo.	14 828	1.5	1.1		69	R.M.	
894	Pecos, Roswell, N. Mex.	14 840	3.75	1904		72	M.V.	
895	Alabama, Selma, Ala.	15 400	9.5			1	S.E.	
896	North Platte, Guernsey, Wyo.	16 200	1.9			69	R.M.	
897	North Pacific Drainage, Calif.	16 543		50	72	70	N.W.	
898	Colorado (Grand), Fruita, Colo.	16 800	3.81	1909		72	R.M.	
899	Mississippi, Anoka, Minn.	17 100	2.87	2		25	M.V.	
900	Gila, Florence, Ariz.	17 750	7.5	1891		72	R.M.	
901	San Joaquin Drainage, Calif.	18 178		30	39	70	S.W.	
902	Missouri, Cascade, Mont.	18 300	2.70	1908	1.3	72	R.M.	
903	White, Clarendon, Ark.	19 000	16.84	1916	8	89	M.V.	
904	Ohio, Pittsburgh, Pa.	19 100	22.98	1907		33	M.V.	
905	Big Horn, Hardin, Wyo.	20 700	1.97	1908	1.4	72	R.M.	
†906	Tennessee, Chattanooga, Tenn.	21 382	34.37	1867	11	1	M.V.	
907	Sacramento Drainage, Calif.	21 420		46	66	70	S.W.	
908	Republican, Bostwick, Nebr.	22 300	1.10			14	M.V.	
909	Snake, South Fork, Minidoka, Idaho.	22 600	2.4		1.9	69	R.M.	
910	Clark Fork, Newport, Wash.	24 000	6.45		4.3	69	N.W.	
911	Susquehanna, Harrisburg, Pa.	24 030	30.60	1889	12	1	N.E.	
912	North Platte, Camp Clarke, Nebr.	24 800	0.95		0.7	14	M.V.	
913	Red, Grand Forks, N. Dak.	25 000	1.70	1897	1.2	25	M.V.	
†914	Republican, Junction, Kans.	25 837	1.80	1903	0.7	72	M.V.	
915	Susquehanna, McCall's Ferry, Pa.	26 766	25.1	1904		69	N.E.	
916	Illinois, at Mouth, Ill.	27 914	4.48	1904		90	M.V.	
917	Rio Grande, San Marcia, N. Mex.	28 067	1.17			84	M.V.	
918	Tennessee, Florence, Ala.	30 800	16.2	1897		72	M.V.	
919	Brazos, Waco, Tex.	30 800	4.3		2	69	M.V.	
†920	Colorado, Austin, Tex.	34 200	3.57	1900	2	72	M.V.	
†921	Ohio, Wheeling, W. Va.	34 800	18.10	1913	11	90	M.V.	
922	Mississippi, St. Paul, Minn.	36 085	3.32	1881	1.7	25	M.V.	
923	Green, Greenriver, Utah	38 200	1.6		1	69	R.M.	
924	Mississippi, Prescott, Wis.	44 070	2.50			30	M.V.	
925	Gila, Yuma, Ariz.	56 000	3.93, Jan.	1916	2	4	75	R.M.
926	Platte, near Columbus, Nebr.	56 900	0.83	1905	0.4	14	M.V.	
927	Kansas, Lawrence, Kans.	59 841	3.80	1903	1	1	M.V.	
928	Ohio, Cincinnati, Ohio.	75 800	8.70	1913		90	M.V.	
929	Muskingum, Marietta, Ohio.	78 500	32.00, Mar.	1913		90	M.V.	
930	Mississippi, Clayton, Iowa.	79 040	2.66	1880		30	M.V.	
931	Ohio, Louisville, Ky.	90 600	8.5	1913		72	M.V.	
932	Red, Arkansas.	97 000	2.32			72	M.V.	
933	Yukon, Eagle, Alaska.	122 000	2.08	1911		72	A.	
934	Colorado, above Gila Junction, Ariz.	169 000	0.89	1909	0.6	0.9	6	R.M.
935	Arkansas, Little Rock, Ark.	170 000	2.7	1916	1.35		89	M.V.
936	Mississippi, Grafton, Ill.	171 570	2.10	1888		30	M.V.	
937	Arkansas and White, Arkansas	189 000	0.84			72	M.V.	
938	Ohio, Paducah, Ky.	205 750	7.00			36	M.V.	
939	Colorado, below Gila Junction, Yuma, Ariz.	225 000	1.05, Jan.	1916		95	R.M.	
940	Ohio, Cairo, Ill.	238 000	6.0	1913	3.6	90	M.V.	
941	Columbia River, Dalles, Ore.	237 000	5.87	1894	3	72	N.W.	
942	Missouri, Sioux City, Iowa	323 462	1.64	1881		30	M.V.	
943	Missouri, St. Charles, Mo.	530 810	1.13			30	M.V.	
944	Mississippi, St. Louis, Mo.	702 880	1.28	1885		80	M.V.	
945	Mississippi, Cairo, Ill.	902 900	2.23	1912	1.3	89	M.V.	
946	Mississippi, Columbus, Ky.	930 540	1.59	1858		72	M.V.	
947	Mississippi, Helena, Ark.	1 000 000	2.04	1912	1.1	89	M.V.	
948	Mississippi, above Arkansas Junction, Ark.	1 050 000	2.31	1912	1.3	89	M.V.	
949	Mississippi River, Mississippi	1 244 000	1.19			72	M.V.	
950	Mississippi, Carrollton, La.	1 400 000	1.07, May,	1922	0.7	89	M.V.	

† Maximum daily flow.

TABLE 2.—(Continued).

FOREIGN RIVERS.								
No.	River or tributary, and location.	Drainage area, in square miles.	RUN-OFF, IN SECOND- FEET PER SQUARE MILE.				Main authority, see Appendix II.	Location.*
			Observed.	Expected peaks.		Maximum and date.		
				Frequent.	Rare.			
1	Tansa, India.....	52.5	666.7				93	F.
2	Ineis, Lanhan, Germany.....	187.4	161.2	July, 1897			46	F.
3	Moselle, Epinal, France.....	313.0	90.4				47	F.
4	Krishna, India.....	345.0	342.6				35	F.
5	Kiewa, Kiewa, Australia.....	709.	3.8	Feb., 1900			35	F.
6	Rhone, St. Maurice, Switzerland.....	1 812.0	17.4		1856		44	F.
7	Mitta, Tallangatta, Australia.....	2 400.0	6.4	Jan., 1903			35	F.
8	Rhine, Lake Constance, Switzerland.....	2 555.0	48.4				55	F.
9	Ovens, Wangaretta, Australia.....	3 000.0	3.3				35	F.
10	Brisbane, Brisbane, Australia.....	5 300.0	10.6	Feb., 1893			35	F.
11	Glommen, Elvrum, Norway.....	5 650.0	15.63				60	F.
12	Goulburn, Murchison, Australia.....	9 000.0	3.6	Mar., 1902			35	F.
13	Weser, Baden, Germany.....	14 640	11.1	Jan., 1841			41	F.
14	Seine, Paris, France.....	16 859	5.24	Jan., 1910			51	F.
15	Lachlan, Forbes, Australia.....	20 159	1.5		1902		35	F.
16	Hun River, China.....	21 000	5.0		1917		92	F.
17	Po, Pontelagoscuro, Italy.....	27 027	9.10				52	F.
18	Danube, Vienna, Austria.....	39 212	8.86	Aug., 1897			50	F.
19	Huai River, China.....	52 000	5.0				92	F.
20	Fitzroy, Australia.....	58 000	10.56	Feb., 1896			35	F.
21	Elbe, Altengamin, Germany.....	60 600	2.15				55	F.
22	Irrawaddy, India.....	149 800	12.9				93	F.
23	Yellow River, China.....	305 000	1.0				92	F.
24	Ganges, India.....	367 970	4.9				93	F.
25	Yang-tze-kiang, China.....	1 100 000	2.73				92	F.
26	Nile, Assuan, Egypt.....	1 300 000	0.35				93	F.
27	Amazon, Obidos, Brazil.....	1 945 000	3.5				96	F.
28	Amazon at mouth, Brazil.....	2 368 000	3.0				96	F.

NOTES.—In addition to the 978 maximum recorded or expected flood rates from Table 2, Plate VIII includes other points as follows:

From Reference No. 98, published subsequently to the completion of Table 2, there are twenty-five flood rates designated (+) for Montana Streams, and not otherwise listed.

From the discussion by the late Emil Kitchling, M. Am. Soc. C. E., of the paper entitled, "Flood Flows", by Weston E. Fuller, M. Am. Soc. C. E. (Reference No. 69), there are platted fifty flood rates designated (F) for European streams not otherwise listed.

In several instances, points with different designations were found to coincide, requiring either a slight displacement or the omission of one or more.

Reference No. 70, *Bulletin No. 5*, "Water Resources of California", includes several streams for which records are incomplete, but the expected maximum flood rates have been carefully estimated; these platted points are distinguished from those regularly listed.

Initials (Table 2), or characters (Plate VIII), to designate each country or section have been selected with regard to their suggestive value.

APPENDIX II

LIST OF AUTHORITIES REFERRED TO IN TABLE 2

- Nos. (1) to (65) inclusive, *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 662, Authorities as listed and cited by the late Emil Kuichling, M. Am. Soc. C. E.
- No. (66) U. S. Geological Survey, *Water Supply Paper No. 426*, "Southern California Floods of January, 1916".
- No. (67) U. S. Geological Survey, *Water Supply Paper No. 488*, "The Floods in Central Texas, September, 1921".
- No. (68) U. S. Geological Survey, *Water Supply Paper No. 487*, "Arkansas River Flood of June 3-5, 1921, Pueblo, Colo."
- No. (69) *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564, "Flood Flows", by Weston E. Fuller, M. Am. Soc. C. E.
- No. (70) *Bulletin No. 5*, "Water Resources of California; Report to the Legislature of 1923 by State Engineering Department".
- No. (71) *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 135, "The Design of Earth Dams", by Joel D. Justin, M. Am. Soc. C. E.
- No. (72) American Civil Engineers' Handbook, 1920 Edition, p. 1170 *et seq.*, Mansfield Merriman, M. Am. Soc. C. E.
- No. (73) *Engineering News-Record*, Vol. 88, 1922, p. 771.
- No. (74) "Observations and Estimates", by C. S. Jarvis, M. Am. Soc. C. E.
- No. (75) *Engineering Record*, Vol. 74, 1916, p. 346.
- No. (76) *Engineering News-Record*, Vol. 89, 1922, p. 921.
- No. (77) " " " Vol. 89, 1922, p. 402.
- No. (78) " " " Vol. 89, 1922, p. 239.
- No. (79) *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1534 *et seq.*, "Flood Problems: A Symposium"; Data on Canadian Rivers by J. B. Challies and W. H. Breithaupt, Members, Am. Soc. C. E.
- No. (80) *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1383 *et seq.*, "Flood Problems: A Symposium".
- No. (81) *Engineering News*, Vol. 74, 1915, p. 742.
- No. (82) *Bulletin*, American Railway Engineering Association, August, 1915.
- No. (83) Water Supply Commission of Pennsylvania.
- No. (84) *Engineering Record*, Vol. 53, 1906, p. 170.
- No. (85) *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 638, Discussion by Herbert E. Bellamy, Assoc. M. Am. Soc. C. E.
- No. (86) *Engineering News-Record*, Vol. 89, 1922, p. 345.
- No. (87) Tenth Biennial Report (Hydrography), State Engineer of Utah, 1916.
- No. (88) *Engineering News-Record*, Vol. 90, 1923, p. 891.
- No. (89) *Engineering News-Record*, Vol. 90, 1923, p. 112, "Hydrologic Record of Mississippi River Floods" by E. N. Chisolm, Jr., M. Am. Soc. C. E.

- No. (90) "Relief from Floods", by Alvord and Burdick.
- No. (91) *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1528, Discussion of "Flood Problems", by H. P. Eddy, M. Am. Soc. C. E.
- No. (92) *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1405, Discussion on "Flood Problems" by John R. Freeman, Past-President, Am. Soc. C. E.
- No. (93) *Engineering News*, Vol. 62, 1909, p. 315, "The Recent Floods at Monterey, Mexico", by G. R. G. Conway, M. Am. Soc. C. E.
- No. (94) *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1383 *et seq.*; various minor references to Symposium on "Flood Problems".
- No. (95) Senate Document 142, Report by A. P. Davis, Past-President, Am. Soc. C. E., Director, U. S. Reclamation Service, on "Development of the Imperial Valley", to 67th Congress.
- No. (96) Encyclopedias: Britannica, International, *et al.*
- No. (97) *Engineering News-Record*, Vol. 83, 1919, p. 28, "Spillway Capacities" by J. T. Whistler, M. Am. Soc. C. E.
- No. (98) *Engineering News-Record*, Vol. 91, 1923, p. 1016, "Flood Flows of Montana Streams", by G. H. Ellis.

THE DESIGN OF A MULTIPLE-ARCH SYSTEM AND PERMISSIBLE SIMPLIFICATIONS

Discussion*

BY MESSRS. HARDY CROSS AND HAROLD B. HAMMILL.

HARDY CROSS,† M. A. M. Soc. C. E. (by letter).‡—This paper presents in somewhat extended form, the so-called “theory of the ellipse of elasticity” and applies it to the analysis of a continuous arch system under the attractive title of “Design and Permissible Simplifications”.

Paper Treats Design in a Restricted Sense.—As the writer commonly understands the term, “design” is the choice of form and arrangement of parts of a structure, under the limitations that practical considerations impose on the values and disposition of loads and other stress-producing conditions. The mathematical determination of theoretical stresses he commonly calls “analysis”. Thus interpreted, a study of the design of multiple-arch systems would at least involve (1) the economic rise ratio in the arch; (2) the economic proportions of the piers; (3) the proper span length; (4) the effect on the choice of working stresses of liability to foundation settlement, probability of simultaneous maximum stresses from foundation effects, temperature, shrinkage, and live load, and especially of the general reliability of this type of structure; and (5) the acceptance or rejection of “split” loading in which the live load is not continuous.

The Author's Thesis.—Evidently, the author interprets “design” more in the sense in which the writer uses “analysis”, for he states that the aim of the paper is to show whether, for practical purposes, it is possible to simplify the “strictly theoretical method” of design. Further, he merely states that the arch was first considered as having fixed ends, “the design being made accordingly”, and the analysis is then extended to include the effect of flexible supports. About half of the paper proper deals with the fixed arch. The author further states that the paper is restricted to a consideration of effects from live load.

Having proportioned one of the multiple arches as a fixed arch, Mr. Janni proceeds to draw influence lines for moments at the kern points for:

- (a) This arch considered as fixed-ended;
- (b) This arch considered as the middle arch of three, the flanking arches being fixed at their outer ends, the load, however, being on the central arch only; and
- (c) The hypothetical flanking arches, the load again being on the central arch only.

* Discussion of the paper by A. C. Janni, M. Am. Soc. C. E., continued from November, 1924, *Proceedings*.

† Prof. of Structural Eng., Univ. of Illinois, Urbana, Ill.

‡ Received by the Secretary, October 1, 1924.

He then states that "from the results obtained in this investigation"—that is, from these influence lines—"the following conclusions seem to be justified." These conclusions are, apparently, that:

- 1.—Professor Guidi's simplification of considering only three arches at a time, and neglecting what lies beyond, is interpreted to mean considering the outer ends of the flanking arches as fixed-ended;
- 2.—That, if the arches in the series, "no matter how many", satisfy an analysis based on this assumption and on "the conditions of loading as shown in this paper", they will be safe.

The author's phrase "conditions of loading as shown" is very obscure. The influence lines on Plate III* are described first as being for the central arch, A_2 , and, later, as being for the side arch, A_1 . If an influence line for the crown moment on A_2 be drawn for the system as shown and if then an influence line for the crown moment on A_1 or A_3 be drawn for the same system, these lines will not be alike. Subsequently, the author separately discusses the design for the main arch and for the flanking arch; that is, he computes different crown moments for two identical adjacent spans of a series—an anomalous condition. The numerical discrepancy is not great, but the obfuscation of Plate III is complete. The diagram is especially confusing to any one who thinks of an influence line as a deflected load line. Perhaps the lines may be clarified by transposing the parts of the diagrams under A_1 and A_3 , considering the whole line as applying to A_2 , and confining the discussion to live load stresses in the central arch. Presumably the author intends that integration under these lines will give maximum kern moments for live load, and thus will solve the problem "in a manner which, in the present state of knowledge, represents the most complete solution that can be desired".

The writer thinks that Professor Guidi's assumption, as stated in the paper, does not necessarily involve fixed-endedness for the flanking arches; that, thus interpreted, it is incorrect, and is dangerous as regards live load stresses in the arch and, in the particular case treated, is on its face highly improbable; and that the acceptance or rejection of "split-load" conditions vitally affects the analysis of continuous arches. Furthermore, he finds in this paper no evidence supporting the assumptions made by its author.

The Ellipse of Elasticity.—Except for the material referred to, the paper is devoted entirely to an exposition of the so-called "theory" of the ellipse of elasticity, the material being much the same as that presented to American engineers by the author on several other occasions. That this method has not become popular with American structural engineers is doubtless explained in part by the brevity of American college courses in analytic geometry—college graduates know little about the properties of poles, polars, and anti-poles and are, perhaps, none the worse for the deficiency. As here presented, the method appears as a highly ornamental and somewhat expensive but not especially useful tool for a structural engineer's already crowded workshop. Although probably historically connected with conceptions of the neutral point, of elastic weights, their statical moments and products of inertia, and with the develop-

* *Proceedings*, Am. Soc. C. E., August, 1924, p. 763.

ment of familiar properties of equilibrium polygons and their use in determining first and second moments, it is not essential to the application of these conceptions and properties. The use of the ellipse in solving a fixed arch seems to present no advantages; and, in the study of continuous arches, it is somewhat inferior to algebraic analysis in flexibility.

The graphical treatment of moments and products of inertia by means of the ellipse of inertia presents some interesting constructions for studying elastic distortions, especially in continuous girders and arches, just as it does for analyzing stresses in unsymmetrical columns. Its pictorial character appeals to some students, while others consider that its demonstrations require the acquisition of an elaborate paraphernalia for which they can ill afford the time. The writer feels that the beauty of this construction has been somewhat marred in presentation by the author's excursion into a discussion of the properties of equilibrium polygons and by digressions such as the proof of the reciprocal relations, where the use of the ellipse is sadly at a disadvantage compared with the direct use of the principle of virtual work. The writer, by the way, wonders at the author's use of the term "virtual work" in Paragraph 13* and Paragraph 19† of the Appendix, where the work seems very real.

The author states that the application of the ellipse to trusses is simpler than to a solid rib and that the latter application depends on the former. This is not clear; indeed the discussion of trusses in this paper seems entirely irrelevant. In this connection the author states that "in most engineering problems it is not necessary to take into consideration the deformations of the web members of a system". In the computation of deflections, the neglect of the web will frequently result in errors of 20% or more. In all important statically indeterminate trusses, except perhaps swing bridges, the final analysis should always include the effect of the web; for approximate analyses, solutions by elastic weights are, in general, not so accurate, so simple, nor so expeditious as by the theorems of beam analysis. The omission of the effect of the web members by Mr. Janni was the main point of reference to the ellipse of elasticity by George F. Swain, Past-President, Am. Soc. C. E., cited in the footnote on page 722.‡

Algebraic Analysis.—General Formulas.—The writer has studied by algebraic methods the properties of the arch system shown, following as nearly as possible the author's method of reasoning, although he does not consider this the simplest method of attack.

Consider any arch and pier, ABC , as shown in Fig. 24. It is desired to determine the elastic properties of the system for forces and moments acting at B . These elastic properties will be defined as;

1.—The co-ordinates of the neutral point, which may be defined as that point which, if rigidly attached to B , will suffer no rotational displacement due to forces acting through it. (This is the center of the author's ellipse of inertia.)

* *Proceedings*, Am. Soc. C. E., August, 1924, p. 781.

† *Loc. cit.*, p. 783.

‡ *Proceedings*, Am. Soc. C. E., August, 1924.

an angle, $\alpha = \tan^{-1} \frac{Z_a}{I_a + I_p}$, and equals $\frac{H_a}{\cos \alpha}$. Also, the horizontal movements of the two points are the same and, following the system of designating displacements indicated, $d H_s = d H_p = H_p J_p = d H_a = \frac{H_a}{\cos \alpha}$ (product of inertia for the arch about this inclined axis and about the horizontal axis),

$$= H_a \left(J_a - \frac{Z_a^2}{I_a + I_p} \right)$$

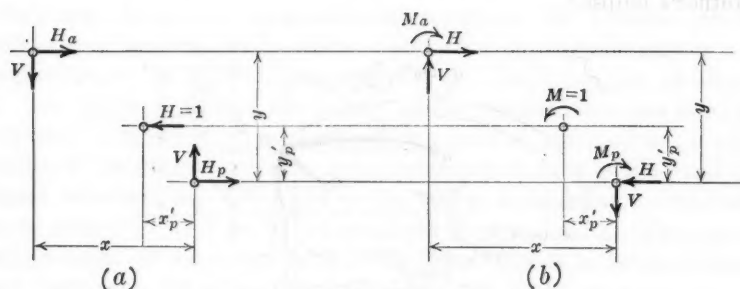


FIG. 25.

Let this new quantity,

$$J_a - \frac{Z_a^2}{I_a + I_p} = J_a'$$

Then,

$$\frac{H_a}{H_p} = \frac{J_a'}{J_p}$$

By statics,

$$H_a + H_p = 1$$

Hence,

$$d H_s = J_s = \frac{J_a' J_p}{J_a + J_p}$$

$$\frac{1}{J_s} = \frac{1}{J_a'} + \frac{1}{J_p}$$

Again, by statics,

$$\frac{y_a'}{y_p'} = \frac{H_p}{H_a} = \frac{\frac{1}{J_p}}{\frac{1}{J_a'}}$$

$$y_a' + y_p' = y'$$

$$y_p' = \frac{J_s}{J_a'} y'$$

Also,

$$Z_x = V I_p = H_a \tan \alpha I_p = I_p \frac{J_s}{J_a'} \tan \alpha$$

By an exactly similar method of reasoning:

$$\frac{1}{I_s} = \frac{1}{I_p} + \frac{1}{I_a'}$$

Where,

$$I_a' = I_a - \frac{Z_a^2}{J_a + J_p}$$

$$x_p' = \frac{I_s}{I_a'} x'$$

Where,

$$x' = x + y \frac{Z_a}{J_a + J_p}$$

Now, apply at s a unit moment (Fig. 25 (b)). This will produce at s a rotation, W_s , but no translation. It will be resisted by four couples, M_a , M_p , H_y , and V_x .

As the three points rotate together,

$$M_a W_a = M_p W_p = W_s$$

$$M_a = \frac{W_s}{W_a}; M_p = \frac{W_s}{W_p}$$

As at x there is rotation only,

$$d H_p = H J_p = y_p' W_s$$

$$H = \frac{y_p'}{J_p} W_s$$

$$d V_p = V I_p = x_p' W_s$$

$$V = \frac{x_p'}{I_p} W_s$$

By statics,

$$M_a + M_p + H_y + V_x = 1$$

$$\frac{W_s}{W_a} + \frac{W_s}{W_p} + \frac{y y_p'}{J_p} + \frac{x x'}{I_p} = 1$$

$$\frac{1}{W_s} = \frac{1}{W_a} + \frac{1}{W_p} + \frac{x x_p'}{I_p} + \frac{y y_p'}{J_p}$$

and the resisting couples are proportional to the four quantities on the right of the equation.

These reduction formulas are perfectly general, since it is always possible to choose the axes of x and y so that they are the principal axes of one of the branches, for which $Z = 0$.

Since the expression for W_s when I_p or $J_p = 0$, is indeterminate, it is permissible to write:

$$\frac{y_p'}{J_p} = \frac{y'}{J_a + J_p}$$

$$\frac{x_p'}{I_p} = \frac{x'}{I_a' + I_p}$$

$$\begin{aligned}\frac{x x_p'}{I_p} + \frac{y y_p'}{J_p} &= \frac{x x'}{I_a' + I_p} + \frac{y y'}{J_a' + J_p} \\ &= \frac{x \left(x + y \frac{Z_a}{J_a + J_p} \right)}{I_a + I_p - \frac{Z_a^2}{J_a + J_p}} + \frac{y \left(y + x \frac{Z_a}{I_a + I_p} \right)}{J_a + J_p - \frac{Z_a^2}{I_a + I_p}} \\ &= \frac{x^2}{I_a + I_p} + \frac{(y')^2}{J_a' + J_p} = \frac{(x')^2}{I_a' + I_p} + \frac{y^2}{J_a + J_p}\end{aligned}$$

If $I_p = 0$, as assumed subsequently,

$$\frac{1}{W_s} = \frac{1}{W_a} + \frac{1}{W_p} + \frac{x^2}{I_a} + \frac{(y')^2}{J_a' + J_p}$$

Although it is not necessary to do so, it is convenient to take $I_p = 0$. This is equivalent to neglecting rib-shortening (direct compression) in the pier. Data are presented later to show the effect of this assumption in the case discussed by the author.

Summarizing, when $I_p = 0$,

$$\begin{aligned}y' &= y + x \frac{Z_a}{I_a} \\ J_a' &= J_a - \frac{Z_a^2}{I_a} \\ \frac{1}{J_s} &= \frac{1}{J_p} + \frac{1}{J_a'} = \frac{J_a' J_p}{J_a' + J_p} \\ I_s &= 0 = Z_s \\ y_p' &= \frac{J_s}{J_a'} y'\end{aligned}$$

$$\frac{1}{W_s} = \frac{1}{W_a} + \frac{1}{W_b} + \frac{x^2}{I_a} + \frac{(y')^2}{J_a' + J_p}$$

Critical Study of the Author's Structure.—The paper gives the following data for the pier and the fixed-ended arch:

$$W_a = 373.5$$

$$I_a = \rho_v^2 W_a = (12.90)^2 \times 373.5 = 62\,100$$

$$J_a = \rho_h^2 W_a = (1.30)^2 \times 373.5 = 630$$

$$Z_a = 0 \quad x = 25.67 \text{ ft.} \quad Y = 18.87 \text{ ft.}$$

$$W_p = 74.6$$

$$I_p = 74.6 \times (0.50)^2 = 18.7$$

$$J_p = 74.6 \times (10.90)^2 = 8\,900$$

$$Z_p = 0$$

As the arch is symmetrical, $Z_a = 0$; $x = x'$; and $y = y'$. Therefore,

$$\frac{1}{W_s} = \frac{1}{373.5} + \frac{1}{74.6} + \frac{(25.67)^2}{62\,100 + 18.7} + \frac{(18.87)^2}{8\,900 + 630} = \frac{1}{15.65}$$

$$J_s = \frac{8\,900}{8\,900 + 630} 630 = 588; \quad I_s = \frac{62\,100}{62\,100 + 18.7} 18.7 = 18.9$$

$$y_p = \frac{588}{630} 18.87 = 17.6; \quad x_p = \frac{I_p}{I_a + I_p} x = \frac{18.87}{62\ 119} 25.67 = 0.008$$

The author gives:

$$W_s = 15.55 \text{ (average of 15.50 and 15.60)}$$

$$J_s = 15.55 \times 38.10 = 593$$

$$I_s = 15.55 \times 1.19 = 18.50$$

$$y_p = 17.596; \quad x_p = 0.007$$

This equivalent pier may now be combined with the next arch (see Fig. 26) disregarding I_p which, as is obvious from the previous computation, has a negligible effect.

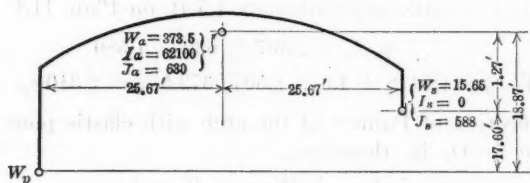


FIG. 26.

The centroid of W_a and W_s lies:

$$\frac{15.65}{373.5 + 15.65} 25.67 = 1.03 \text{ ft. to the right of } W_a \text{ and}$$

$$\frac{15.65}{373.5 + 15.65} 1.27 = 0.05 \text{ ft. below } W_a$$

$$W_t = W_a + W_s = 373.5 + 15.65 = 389.2$$

$$I_t = I_a + \frac{W_a W_s}{W_a + W_s} (25.67)^2 = 62\ 100 + 9\ 900 = 72\ 000$$

$$J_t = J_a + J_s + \frac{W_a W_s}{W_a + W_s} (1.27)^2 = 630 + 588 + 24 = 1\ 242$$

$$Z_t = \frac{W_a W_s}{W_a + W_s} 25.67 \times 1.27 = 490$$

$$x = 25.67 + 1.03 = 26.70; \quad y = 18.87 - 0.05 = 18.82$$

These are the values of the new arch which is to be combined with the pier, as follows:

$$y' = 18.82 + 26.70 \frac{490}{72\ 000} = 18.99$$

$$J_a' = 1\ 242 - \frac{(490)^2}{72\ 000} = 1\ 242 - 3 = 1\ 239$$

$$\frac{1}{W_s} = \frac{1}{389.2} + \frac{1}{74.6} + \frac{(26.70)^2}{72\ 000} + \frac{(18.99)^2}{8\ 900 + 1\ 239} = \frac{1}{16.28}$$

$$J_s = \frac{8\ 900}{8\ 900 + 1\ 239} 1\ 239 = 1\ 085; \quad y_p = \frac{1\ 085}{1\ 239} 1\ 899 = 16.65$$

Study of Crown Thrust.—A comparison of the two sets of computations just made, shows that if the series is taken as five arches instead of three, very

different results obtain. A further study will show what effect this will have on the horizontal thrust at the crown of the middle arch due to a unit load at this point. A unit horizontal force at a (Fig. 27) will produce a vertical deflection of $b = i_h J_a$, where i_h is the influence ordinate at b for the horizontal thrust on the fixed arch. Also, a unit moment at a will produce a vertical deflection at b equal to $i_m W_a e_y$, in which, i_m is the influence ordinate at b for the moment at the neutral point of the fixed arch. In Table 3,* the author gives $i_h = 2.507$. The influence ordinate for the moment at B is given as:

$$\frac{1.980 + 2.857}{2} = 2.418 \text{ (Table 3)}$$

and the distance, $a b$, scales approximately 1.5 ft. on Plate II.† Hence,

$$i_h J_a = 2.507 \times 630 = 1580$$

$$i_m W_a e_y = (2.418 + 1.5 \times 2.507) 373.5 e_y = 2310 e_y$$

The vertical deflection of Point b of the arch with elastic piers, due to a unit horizontal force at O , is, therefore,

$$i_h J_a - i_m W_a e_y + W_x x d$$

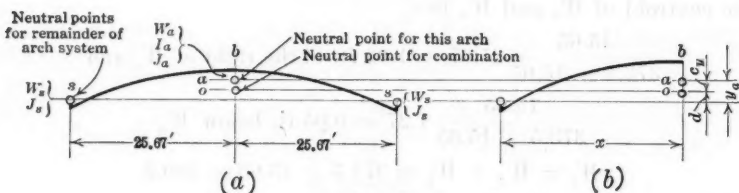


FIG. 27.

The new influence ordinate for H is the quotient of this value and the moment of inertia about the horizontal axis through O of the combination, which will be called J_O .

Applying this to the combinations found previously, if,

$$W_s = 15.65, J_s = 588, y_a = 1.27$$

$$e_y = \frac{2 \times 15.65}{373.5 + 2 \times 15.65} 1.27 = 0.10$$

$$d = 1.27 - 0.10 = 1.17$$

$$i_h J_a + W_s x d - i_m W_a e_y = 1580 + 15.65 \times 25.67 \times 1.17 - 2310 \times 0.10 = 1825$$

$$J_O = J_a + 2 J_s + \frac{2 W_s W_a}{W_a + 2 W_s} (1.27)^2 = 630 + 1176 + 47 = 1853$$

$$i_h' = \frac{1825}{1853} = 0.99$$

In Table 7‡ the author gives $i_h' = 1.009$.

* *Proceeding, Am. Soc. C. E., August, 1924, p. 761.*

† *Loc. cit., p. 757.*

‡ *Loc. cit., p. 768.*

If, however, a series of five arches is taken:

$$W_s = 16.28; J_s = 1\ 085; y_a = 18.85 - 16.65 = 2.22$$

$$e_y = \frac{2 \times 16.28}{373.5 + 2 \times 16.28} 2.22 = 0.18$$

$$d = 2.22 - 0.18 = 2.04$$

$$i_h J_a + W_s x d - i_m W_a e_y = 1\ 580 + 16.28 \times 25.67 \times 2.04 \\ - 2\ 310 \times 0.18 = 2\ 057$$

$$J_O = J_a + 2 J_s + \frac{2 W_a W_s}{W_a + 2 W_s} (2.22)^2 = 2\ 947$$

$$i_h' = \frac{2\ 057}{2\ 947} 0.697$$

The writer has extended this investigation, including, successively, three, five, seven, nine, etc., arches in the series (the ends of the series being fixed), and has determined the effect on the influence ordinate for H at the center of the middle arch. The data are given in Table 9.

TABLE 9.—VARIATION IN CERTAIN PROPERTIES OF THE MIDDLE ARCH ACCORDING TO THE NUMBER OF ARCHES IN THE SYSTEM.

Number of arches considered.	Rotational resistance of remainder of series, W_s .	Horizontal resistance of remainder of series, J_s .	Location of center of rotation of remainder of series, y_a , in feet.	INFLUENCE ORDINATE FOR THRUST AT CROWN OF MIDDLE ARCH DUE TO LOAD THERE, i_h .		
				True value.	Author's value.	Author's error, percentage.
1	2.507
3	15.65	588	1.27	0.99	1.009	0
5	16.28	1 085	2.22	0.697	1.009	45
7	16.65	1 489	2.92	0.560	1.009	80
9	17.00	1 815	3.57	0.517	1.009	95
11	17.25	2 022	3.99	0.465	1.009	120
13	17.38	2 170	4.27	0.450	1.009	124

It is evident that the properties of the remainder of the series of arches on each side are represented by convergent series which, in the case selected by the author, converge slowly. This is due to the extremely slender piers used in the example chosen. In the extreme case of piers offering only vertical resistance (horizontal resistance, nil), the value of J_s , which measures the horizontal movement of one end of a series of arches due to horizontal loading is directly proportional to the number of arches in the series. The author, however, assumes that, in cases approaching this, the horizontal movement is independent of the number of arches in the series.

The Author's Structure not Representative.—The example taken by the author seems unfortunate in that the arches are so flat and the piers offer so little resistance to horizontal movement that the case approaches that of a continuous girder, as far as the stresses in the arch are concerned. By summing the influence ordinates, as contained in the paper, for a load, W , uniformly distributed over the middle arch: The crown thrust for an arch on piers = $0.653 W$. Taking the scaled distance, 1.5 ft. from the crown to the

neutral point, the moment at the crown, due to the thrust (if on piers) = $1.5 \times 0.653 W = 0.979 W$. Taking a live load of 150 lb. per sq. ft. ($W = 50 \times 150$ lb. per ft. of width), the reduction in crown moment for a 1-ft. strip of arch ring (assuming a barrel arch) equals:

$$50 \times 150 \times 0.979 = 7\,350 \text{ ft.-lb.}$$

The temperature moment due to a change of 50° equals:

$$\frac{50^\circ \times 0.000006 \times 50 \text{ ft.} \times 1\,500\,000 \times 144}{630} 1.5 = 7\,700 \text{ ft.-lb.}$$

By fixing against horizontal movement the ends of these arches, the loss in temperature stresses is about as much as the gain—even on the author's assumptions—in live load stresses. Any economy would be in the dead load stresses; to erect such a series if it were many spans in length without sacrificing this economy by increasing the size of pier to provide for unbalanced erection thrusts would present a difficult problem. Only strong æsthetic considerations would apparently justify building this series as shown, and even then the average structural engineer would probably design the arch as a continuous girder.

Complete Analysis Practicable.—Finally, the writer believes that a correct and practically complete analysis of a mutiple-arch system is quite feasible and that the amount of labor involved is not unreasonable in the case of an important structure; the correct and economical design of the system is another matter. The writer heartily concurs in the author's statement that:

"The designer should investigate the behavior of a multiple-arch system in every case where the importance of the spans, the comparative slenderness of the piers, and the conditions of the foundation are such as to warrant a thorough investigation."

He would like to emphasize strongly the last two words.

HAROLD B. HAMMILL,* ASSOC. M. AM. SOC. C. E. (by letter).†—American engineers are indeed indebted to Mr. Janni for presenting in English this improved method of analyzing an arch, namely, by the ellipse of elasticity. To the best of the writer's knowledge his presentation‡ of the method was the first published in English.

In association with John B. Leonard, M. Am. Soc. C. E., the writer has been using this method for several years and finds it both quick and accurate. Some engineers object to it as a graphical determination for stresses, claiming that it is, for this reason, less accurate than an analytical one, but the writer has found that analytical and graphical computations by the usual elastic theory will check within 2 per cent.

Of interest to engineers is the greater accuracy obtained by this method. At best, the equations expressing the elastic relations of an arch ring are based on several assumptions. By the determination of the "centers of rotation" of segments of the arch ring, the number of assumptions is reduced by

* Cens. Engr., San Francisco, Calif.

† Received by the Secretary, October 21, 1924.

‡ *Journal*, West. Soc. of Engrs., May, 1913.

one and the problem is brought one step nearer an exact analysis. In some cases, stresses determined in this manner exceed those obtained by the generally accepted elastic theory by percentages that are greatly in excess of those, due to the use of graphics. The writer has found that some arches, considered as fixed-ended, show the stresses determined by the ellipse analysis to be as much as 20% greater than by the usual elastic theory.

This variation is influenced by the ratio of rise to span length as well as by the dimensions of the rib itself. As the ratio of rise to span length increases, the difference in stress decreases; for an arch with a very low ratio, it would be considerable.

The inclusion of the elastic piers in the analysis of a series of arches influences the stresses in proportion to the elasticity of the pier. For any unbalanced loads the stresses in an arch of a series are always greater than those determined from the analysis of the arch alone.

Fig. 28 (a) shows the outline and general dimensions of a structure of three equal spans for which the stresses have been determined by three analyses, the results of which are given in Table 10.

TABLE 10.—MAXIMUM COMPRESSIVE STRESSES, IN POUNDS PER SQUARE INCH, ARCH OF FIG. 28 (a).

	SINGLE FIXED-ENDED ARCH.		THREE-SPAN UNIT WITH PIERS.
	Ordinary elastic method.	Author's method.	Author's method.
At crown.....	660	680	797
At point near crown.....	750	764	828
At haunch.....	825	960	1 025

Fig. 28 (b) shows the outline and general dimensions of a structure designed for a stream in low-lying country. The fact that the distance between the roadway level and high-water elevation was limited, made it desirable to proportion the spans such that the side spans were shorter than the central one. The ratio of rise to span length was kept approximately constant for all spans. In Table 11 are recorded the stresses in the central span computed by the usual elastic method for a fixed-ended arch and by the author's method, including piers.

TABLE 11.—MAXIMUM COMPRESSIVE STRESSES, IN POUNDS PER SQUARE INCH, ARCH OF FIG. 28 (b).

	Ordinary elastic method.	Author's method.
Point near crown.....	692	930
Point near haunch.....	614	928

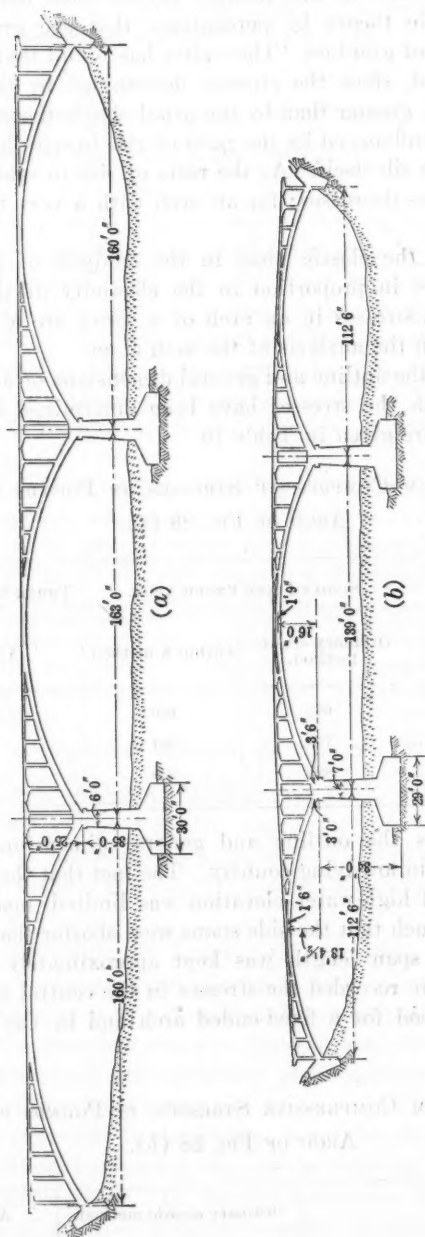


FIG. 28.—ELEVATIONS AND DIMENSIONS OF TRIPLE ARCH BRIDGES.

A designer controlled by pre-established maximum stresses would be considerably in error for such a structure as that shown in Fig. 28 (b), if he accepted the results of the usual fixed-ended rib analysis as correct. The value for the maximum foundation pressure would be too great, as the adjacent arch takes some of the unbalanced thrust and the value for the maximum compression in the rib would be too low.

From experience with bridges of more than three spans the writer can concur with Mr. Janni in his conclusion that the unit of three spans is sufficient for determining the maximum stresses. The relative pier dimensions of those structures which have been investigated, have always been much larger than those of the author's example due to considerations of stiffness. Experience shows that the difference in stress computed by a three-span unit *versus* a five-span unit would not exceed 2 per cent.

WATERWAY AND RAILWAY EQUIVALENTS

Discussion*

By C. E. GRUNSKY, PRESIDENT, AM. SOC. C. E.

C. E. GRUNSKY,† PRESIDENT, AM. SOC. C. E. (by letter).‡—In his opening paragraph the author presents the conclusion that a new waterway line of transportation should not be established unless the need for it can be shown and unless, "if established, it will produce an annual saving in the cost of transportation greater than the interest on construction plus maintenance and operating costs". There is another element which should sometimes be considered when the advisability of such a line of transportation is being studied, and by virtue of which many a project may be found advisable, despite the fact that it cannot, perhaps, for many years in the future, produce revenue to meet in full the interest charges against it. This element is the project's contribution to general prosperity as measured in terms of population, of business, and of property values. It becomes a factor because the expenditure of funds in its construction, and the utilization of the facilities which it provides when it is completed, bring increase of population; and to the extent that the growth in population of the zone coming under the influence of the improvement is thus accelerated, to the same extent will land values increase and wealth be created, which may fairly be weighed against the cost of the improvement when the advisability thereof is under consideration. Society, in other words, may reap material advantage from many an enterprise that would be condemned by the stockholder who weighs only the cost of the service against the prospective revenue.

It is, no doubt, this "benefit to Society" that has led to many large expenditures by the Government, without hope of financial return, for navigation and other projects which could not have been privately or locally financed; and it is this same consideration which in the past history of the country has prompted so many of the bonuses to railroad and other enterprises, by cities, counties, and States, as well as by the United States.

As illustrative of the foregoing proposition the following is taken from a paper entitled "History of the Conversion of the River Clyde into a Navigable Waterway, and of the Progress of Glasgow Harbour from Its Commencement to the Present Day",§ by James Deas, Engineer of Clyde Navigation, read at the International Engineering Congress of the Columbian Exposition, 1893:

* This discussion (of the paper by William M. Black, M. Am. Soc. C. E., published in August, 1924, *Proceedings*, and to be presented at the meeting of December 10, 1924) is printed in *Proceedings*, in order that the views expressed may be brought before the members for further discussion.

† Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

‡ Received by the Secretary, September 22, 1924.

§ *Transactions*, Am. Soc. C. E., Vol. XXIX (1893), p. 129.

"These improvements have raised Glasgow within the short period of 90 years from a second-rate inland provincial town, with a population of 77 000, to be the second city of the Empire, with a population of 658 073, and the chief seaport of the west of Scotland; increased the value of lands on the river's sides from Glasgow seaward a hundred-fold; created the burghs of Govan and Patrick; given wealth to thousands, and the means of life to hundreds of thousands of the inhabitants of the northern portion of Great Britain, and emphasized in a marked degree the local epigram, 'Glasgow made the Clyde and the Clyde made Glasgow.'"

In the same vein, the late W. Henry Hunter, M. Am. Soc. C. E., Chief Engineer to the Manchester Ship Canal, in a paper entitled "Artificial Waterways in Great Britain",* presented at the International Engineering Congress, held at St. Louis, Mo., in 1904, said:

"Apart from some such State aid, it is difficult to see how the scheme can be financed satisfactorily, particularly after the experience of the working of the Manchester Ship Canal, of which it may be truly asserted that such working has been and is the occasion of almost incalculable benefit, both direct and indirect, to all within the sphere of influence of the waterway, except the shareholders who provided the money for the inception of the project and the construction of the greater part of the undertaking." * * *

"* * * The operation of the Manchester Ship Canal has directly or indirectly affected beneficially the whole of the greatest industrial district on the face of the earth; a district of which the present population does not fall far short of 10 000 000 persons; while in the more limited area of which the City of Manchester is the heart and center, many industries have been saved from extinction and many others from decline and ultimate decay". * * *

"Now, not only has decline been arrested, and decay averted, but lines of new and vigorous growth have shot and are shooting out on every side, north, south, east, west, there is no exception; expansion is universal; old industries have been revived and new industries introduced; the change is definite and impressive, and has been due solely to the effect and influence of the canal."

* Transactions, Am. Soc. C. E., Vol. LIV (F) (1905), p. 190.

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THE INTERCONNECTED POWER SYSTEMS OF THE SOUTHEAST

Discussion*

By GEORGE A. ORROK, M. AM. SOC. C. E.

GEORGE A. ORROK,† M. AM. SOC. C. E.—It is well known that the low-water flow of the Tennessee River between Hales Bar and Muscle Shoals varies from 3 500 to 5 000 sec.-ft. The Government requires that the low-water flow fixed at 5 000 sec.-ft. shall be maintained whether or not power is being generated. This would mean at Hales Bar on a 39-ft. head the production of 16 800 kw. and at Muscle Shoals probably 48 000 to 50 000 kw. One hears 200 000 kw. spoken of very glibly, and even 1 000 000 h.p., but these figures do not quite square with the facts. In discussing a water-power plant, even if it includes storage in the head-waters, well known flow records should not be disregarded; and the flow records on the Tennessee River have been kept rather accurately for many years.

Those who have had to do with the development of steam-power plants are also greatly interested in the building of steam plants at coal mines. Unfortunately, for every ton of coal burned in the steam-power station, from 600 to 1 000 tons of water are necessary. Almost invariably this condensing water has to be pumped; there are only one or two exceptions in the United States. In the North 600 tons of water are required per ton of coal, and in the South it would be more like 1 000 tons.

It is better to haul coal on a railroad than to pump water any great distance or attempt to use cooling towers as they do in England or Germany. Recently, the speaker visited a station, built in rather close proximity to the coal mine, that has been forced to use the sewerage discharge of a large city for its supply of condensing water. In order to cool this discharge they have to use towers, so that the station is surrounded by a forest of cooling towers, thirty-seven in number, 35 ft. wide by 100 ft. long. They have rain from the cooling towers every afternoon. There are power-station locations at the coal mine, with a river in close proximity, an ideal location.

The speaker has followed this subject of interconnection for a great many years. It is only of use where diversity exists. There is a wonderful interconnected system in the City of New York, which is useless because there is no diversity. The peaks of demand come within 3 or 4 min. of each other. Whenever one system wants power, the other system also wants it, and there is no chance for interchange except in case of a breakdown. It is true they do interchange power in the New York District to the extent of probably 70 000 or 80 000 kw. of a total load of 1 600 000 kw.

* Discussion of the paper by Charles G. Adsit, M. Am. Soc. C. E., continued from November, 1924, *Proceedings*.

† Cons. Engr., New York, N. Y.

In other cases, there is diversity of load and diversity of water supply, so that interconnection can be of great value. This has been exemplified in the South, in the Pittsburgh District, in the vicinity of New York outside of the Metropolitan District, in the West around Chicago, Ill., on the Pacific slope, in Italy, in Switzerland, and, the speaker understands, in France and Germany. The interconnection of the electrical transmission lines in Northern Italy is of use because of the diversity of the water power, some of the rivers obtaining their water from the melting of the Alpine snows, others from rain falling on the Apennines.

The interconnecting systems in Switzerland are not quite so diversified; they are connected in three places to Germany on the north, two to France on the west, and four to Italy on the south, but to date little power has been transferred. The interior connections are particularly useless in that respect. It is hoped, however, that in the course of a few years more sources of power will be developed and more diversity will be secured. In their interconnected systems along the Belgian line, and also in the south of France, French engineers are attempting to tie in with the distribution system of Paris, and by those connections to secure both diversity of load and supply.

As to interconnection in the Northern United States, it is not generally known that Manlius, N. Y., and Portsmouth, N. H., are already connected. The Adirondack Company, in connection with the Syracuse Company, is building a tie line of 12 miles between Manlius and Syracuse. This will enable the Company to transmit power from Portsmouth to Windsor, Ont., Canada. Power is now being transmitted or interchanged from the Adirondack System to the New England Power System, which includes a stretch practically 500 miles in length.

SECONDARY STRESSES IN BRIDGES

Discussion*

BY MESSRS. O. H. AMMANN, WILLIAM CAIN, ALMON H. FULLER, AND
K. P. BAHJEJIAN.

O. H. AMMANN,† M. AM. SOC. C. E. (by letter).‡—The author deserves credit for his painstaking and elaborate investigation. For practical purposes the outstanding merit of this paper lies in its critical comparison of various methods of calculating secondary stresses.

Contrary to Mr. von Abo the writer does not expect that engineers will avail themselves extensively of a systematic determination of secondary stresses in bridges, nor does he believe that, in general, such practice would produce bridges of greater strength and economy than those built with what the author calls "the factor of ignorance".

An experienced engineer, of course, must be familiar with the character of secondary stresses, but he will not trouble himself with their determination in each ordinary case. For an unusual structure he will generally know where large secondary stresses are to be expected and confine himself to such critical determinations. Nor will it be a question of extreme refinement for he knows that many "practical factors" affect the true values and that even the true secondary stresses are not at all a criterion or measure for the safety or ultimate carrying capacity of a bridge. He will prefer therefore such methods as will determine these stresses, where desirable, in as simple a manner as is compatible with a fair degree of accuracy.

The writer has calculated the secondary stresses in a number of trusses and has invariably used the semi-graphical method of Mohr with which he was most familiar and which appeared to him to have the advantages of simplicity and lucidity.

A modification of this method which appears to possess its advantages to an even greater degree has since been developed by M. Rös and is contained in a report by the Technical Commission of the Swiss Association of Structural Steel Manufacturers on a comprehensive investigation of secondary stresses. A brief résumé of that report is given in a recent article by the writer.§

The author mentions only casually the effect of gusset-plates on the true secondary stresses, yet this is most important and, more than anything else, suggests the futility of extreme accuracy in the theoretical determination of secondary stresses for the purpose of design. He assumes that the theo-

* This discussion (of the paper by Cecil Vivian von Abo, Jun. Am. Soc. C. E., published in September, 1924, *Proceedings*, and presented at the meeting of November 5, 1924), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., New York, N. Y.

‡ Received by the Secretary, October 2, 1924.

§ *Engineering News-Record*, October 23, 1924.

retical secondary stresses exist as sections beyond the end connections. Although that arbitrary assumption is often made, and is, perhaps, as good as any other in many practical cases, it is far from correct where gussets are rigid and comparatively large. The true state is extremely difficult or almost impracticable of determination by theory, but from practical observations, especially those previously mentioned, it appears that the maximum stresses beyond the end connections may be found approximately by multiplying the secondary moments at the theoretical ends of a member by the ratio of the theoretical to the actual length of the member (between connections) and assuming these increased moments to be applied at the points of connection with the gussets.

It is evident that in many cases this correction means considerably greater secondary stresses. It remains an open question, however, as to whether rigid gussets, producing large secondary stresses, are not a source of strength rather than of weakness. The writer is inclined to the view that the former is the case, although he feels that this question offers a field for further investigation.

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).†—This valuable paper on secondary stresses in bridges is unique, in that it gives, not only the theory of various methods pertaining to secondary stresses, but likewise their application to a railway bridge, with the numerical or graphical work required for complete solutions, the main object being a critical comparison of the methods with a view to selecting the best of them, considering the labor involved and the accuracy. This task the author has accomplished successfully, and the full numerical solutions show the comparative merits, from a practical point of view, of the several methods. The author finally concludes, that, for the railroad truss, Manderla's method is to be preferred; the writer is inclined to agree with him, perhaps using as a check the Mohr semi-graphical method. The computations by either of these methods involve only three decimal figures, whereas Müller-Breslau's or Mohr's elastic weight methods introduce a much greater number of equations, which thus require many more decimal places for an accurate solution.

The writer having used the Gauss method of elimination considerably, warmly commends it. By this method, each derived equation is checked at once, so that the computer has the great satisfaction of being assured of correct results at every stage of the elimination. Professor Turneaure‡ has given a complete derivation of the required formulas by Manderla's method, together with a systematic method of procedure for their solution, which is to be commended. Tentative methods, such as those of Ritter and Mao, are not very attractive when direct solutions that require no more labor are available.

In connection with the development of Mohr's elastic weight method, the author states,§ that, after the axes are turned 90° , " $u_{4\psi}$ of Equations (69) may

* Prof. Emeritus, Univ. of North Carolina, Chapel Hill, N. C.

† Received by the Secretary, October 16, 1924.

‡ Johnson, Bryan, and Turneaure, "Modern Framed Structures," Vol. II.

§ *Proceedings, Am. Soc. C. E.*, September, 1924, p. 1001.

be regarded as the negative of the sum of the moments of the ψ 's about the new u' -axis, while $v_{4\phi}$ is the sum of the moments of the ϕ 's about the new v' -axis." Evidently, if the ψ 's are turned to a vertical position, they can have no moments about a horizontal line (the new u' -axis); and, if the ϕ 's are turned to a horizontal position, they can have no moments about a vertical line (the new v' -axis. (See Fig. 93.)

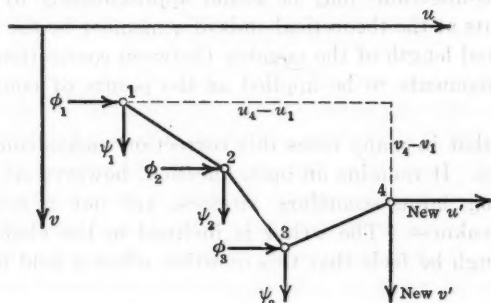


FIG. 93.

From Equations (68):

$$u_{4\psi} = (u_4 - u_1) \psi_1 + (u_4 - u_2) \psi_2 + \dots,$$

$$-v_{4\phi} = -[(v_4 - v_1) \phi_1 + (v_4 - v_2) \phi_2 + \dots]$$

The distances $(u_4 - u_1), \dots, (v_4 - v_1), \dots$ (Fig. 93), are unchanged by the rotation of the axes; but if the ϕ 's are regarded as forces acting horizontally and the ψ 's as forces acting vertically, then taking counterclockwise moments as positive:

$$\begin{aligned} du_4 &= (u_{4\psi} - v_{4\phi}) \\ &= \sum \text{moments of } \psi\text{'s about the new } v'\text{-axis} \\ &\quad + \sum \text{moments of } \phi\text{'s about the new } u'\text{-axis} \end{aligned}$$

Further, the author shows, when $du_4 = 0$, that the origin of the new axes can be taken anywhere, as illustrated in the triangles of Fig. 55,* where u_2, v_2 , correspond to the new u' -axis and the new v' -axis, respectively. Formulas such as II 2 in Table 18† are at once derived by the formula just stated. The author will doubtless clear up the ambiguity mentioned. He is requested to explain (1) why the co-ordinates, u_1, v_1 , etc., are not drawn parallel to the axes, u, v , respectively, as is usual, and (2) as to the signs of the co-ordinates.

This method of elastic weights is readily adapted to the rectangular frame, as the author illustrates with an example. He also gives Batho's interesting analysis,‡ leading to Equations (117), which apply to any rectangular frame, and thus makes possible a solution by mere substitution. The author has failed to note two important papers on statically indeterminate frames, one by Mikishi Abe,§ the other by Messrs. Wilson, Richart, and Weiss.¶ These

* *Proceedings, Am. Soc. C. E.*, September, 1924, p. 1060.

† *Loc. cit.*, p. 1062.

‡ *Loc. cit.*, p. 1104.

§ Univ. of Illinois *Bulletin No. 107*, October 21, 1918, "Analysis and Tests of Rigidly Connected Reinforced Concrete Frames", by Mikishi Abe; and Univ. of Illinois *Bulletin No. 108*, November 4, 1918, "Analysis of Statically Indeterminate Structures by the Slope-Deflection Method", by W. M. Wilson, F. E. Richart, and Camillo Weiss.

papers are especially valuable to the designer, in that they give readily applied formulas for numerous statically indeterminate structures.

In conclusion, the writer desires to express his appreciation of the author's notable contribution, which has evidently involved great labor and which leads to practical conclusions that will prove of material service to engineers.

ALMON H. FULLER,* M. Am. Soc. C. E. (by letter).†—Although this paper may not contain much that is new in the sense of theoretical or experimental research, it is a valuable contribution to the subject of secondary stresses. The procedure of computation and the comparison of methods are of value to all structural engineers, especially to those, still in the majority, who have not given personal attention to this phase of analysis. The emphasis on the effect of lateral and sway-bracing and on the general subject of stress distribution helps greatly in producing a well rounded investigation.

The value of the paper is enhanced by references to, and brief comments on, the field measurements of stresses which were available. Other similar investigations to which the author refers as not being available may be many and, if brought to light, may be co-ordinated so as to be of considerable value. To this end, the writer contributes his mite by giving some significant data which are yet only published in part.

The results for 1922‡ and for 1923§ of a four-year program in stress measurements on bridges have appeared since the paper was written. While the object has been primarily to investigate impact in highway bridges, the study of stress distribution under static and dynamic loads has naturally followed, and although the greater part of the work relates to floor systems, considerable data on truss members have been obtained. For instance, the 1923 results for the hip vertical of the Skunk River Bridge show a unit stress of 6 200 lb. per sq. in. on the inside flanges (the ones to which the floor-beams are attached) and 2 400 lb. on the outside flanges, under an impact of about 25% (live load stresses with the load in motion on a smooth floor exceed by 25% the static stresses for the same load). Under high impact (above 100%), the respective values are 13 000 and 4 800 lb. per sq. in. Corresponding stresses for the hip vertical of the Rowland Bridge are 4 100, 1 700, 8 500, and 5 000 lb. per sq. in. A tendency will be noted in the Rowland Bridge for a smaller proportionate range of stress under the more severe loading. The bending in these members is almost entirely in planes perpendicular to the trusses; there is little indication of secondary stresses due to bending in the planes of the trusses.

The results of some of the writer's strain-gauge readings for secondary stresses on the Niagara Arch|| have already been published. They indicate the bending in horizontal and in vertical planes and are averages of two or more readings. The individual readings, not given in that paper, indicate

* Prof., Civ. Eng., Iowa State Coll.; Cons. Bridge Engr., Iowa State Highway Comm., Ames, Iowa.

† Received by the Secretary, October 20, 1924.

‡ "Preliminary Impact Studies—Skunk River Bridge on the Lincoln Highway near Ames, Iowa," *Bulletin No. 63*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa, and *Proceedings*, Am. Soc. C. E., March, 1923, Papers and Discussions, p. 457.

§ "Impact Tests on Highway Bridges," *Public Roads*, September, 1924.

|| "Revision of the Niagara Railway Arch Bridge", by Charles Evan Fowler, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 1936.

even a greater range of stress; they show that for the upper panel point, U_2 , for a given static live load, G_2 , the range of the stress in the upper chord to the left is from 5 300 lb. per sq. in. compression to 2 800 lb. tension; in the upper chord to the right, from 100 to 4 800 lb. compression; in the vertical, 1 000 to 5 200 lb. compression; and in the diagonal, from 2 800 to 7 000 lb. tension. For the corresponding members at the point, U_3 , for a given static live load, D_1 , the values are, respectively, 3 000 lb. per sq. in. compression to 800 lb. tension, 1 400 to 3 600 lb. compression, 400 lb. tension to 6 200 lb. compression and 2 400 to 4 200 lb. tension.

A bulletin* by the writer, now in press, gives the results of strain-gauge readings at sixteen points on each of three sections of four columns during the course of erection of an eighteen-story building. In general, the distribution of stress was excellent. There was one exception, however, of sufficient range to merit attention, although not to cause alarm. Just above the base of one column under a load of about 700 000 lb., the average stress was about 10 000 lb. per sq. in. and the range from less than 7 000 to more than 14 000 lb. The average of the four lowest points on one side of the column was 7 400 lb. per sq. in., and the average of the four highest corresponding points on the other side of the column was 13 600 lb. This shows an unmistakable bending about an axis very nearly parallel to two sides and suggests something which should be known, and which, if anticipated, might be prevented.

As yet, none of these values has been checked against computed secondary stresses, but, regardless of what the comparison may be, they show such a range as to give encouragement for further investigations from both theoretical and experimental standpoints. It would seem fortunate if field measurements could be taken on the structure for which the author has made such satisfactory computations.

K. P. BAHJEJIAN,† Esq. (by letter).‡—On comparing the methods of Mao and Müller-Breslau it is noted that their fundamental equations are similar in form. If Equation (80)§ is applied to the angle at A in Figs. 7 and 8,|| and the same notation is used as indicated in these diagrams, remembering that f and M have the same subscripts, there is obtained:

$$6 E \cdot \Delta \alpha = \frac{l_1}{y_1} (2 f_2 - f_1) - \frac{l_2}{y_2} (2 f_3 - f_4)$$

The first form of Equation (43)¶ is:

$$6 E \cdot \Delta \alpha = (2 \rho_3 - \rho_4) - (2 \rho_2 - \rho_1)$$

in which,

$$\rho_3 = \frac{M_3 l_2}{I_2} \text{ and } M_3 = \frac{f_3 I_2}{y_2}, \text{ whence } \rho_3 = \frac{f_3 l_2}{y_2}$$

* "Measurements of Stresses in Four Steel Columns of the Equitable Building, Des Moines, Iowa", *Bulletin No. 71*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa.

† Engr. with East Penn Elec. Co., Pottsville, Pa.

‡ Received by the Secretary, November 5, 1924.

§ *Proceedings*, Am. Soc. C. E., September, 1924, p. 1009.

|| *Loc. cit.*, p. 987.

¶ *Loc. cit.*, p. 988.

Similarly,

$$\rho_4 = \frac{f_4 l_2}{y_2}, \rho_2 = \frac{f_2 l_1}{y_1}, \text{ and } \rho_1 = \frac{f_1 l_1}{y_1}$$

Making these substitutions, the equation becomes:

$$6 E \cdot \Delta \alpha = \frac{l_2}{y_2} (2 f_3 - f_4) - \frac{l_1}{y_1} (2 f_2 - f_1).$$

As bending moments in a clockwise direction in the latter method are regarded as positive, and in the former as negative, these equations are really identical.

The author has called attention to the fact that in applying the fundamental equations to a given truss, Mao "assumes two quantities known at the start, and using these throughout the process, finally arrives at two simultaneous equations the solution of which determines the two assumed quantities", just as described in connection with Fig. 9* for Müller-Breslau's procedure modified by Páez. Mao assumes the secondary stresses, f_1 and f_2 , in the first member, 1-2, of the truss, instead of ρ_1 and ρ_2 for the same member.

* *Proceedings, Am. Soc. C. E., September, 1924, p. 989.*

INCREASING THE CAPACITY OF EXISTING STREETS

Discussion*

BY ARTHUR S. TUTTLE, M. AM. SOC. C. E.†

ARTHUR S. TUTTLE,‡ M. AM. SOC. C. E. (by letter).§—Mr. Corbett|| points out that, as New York grows upward, additional street capacity must be provided to carry the corresponding increase in pedestrian and vehicular traffic. His method of securing relief is based on the adoption of the arcade principle, but is carried much further than that proposed by the writer and his argument in its support is regarded as an endorsement of the practicability of introducing arcades without sacrificing the appearance or attractiveness of the arcaded buildings.

The Chicago improvements, discussed by Messrs. Hill and Crane,¶ were designed to meet special conditions and have undoubtedly served their purpose well. The means proposed for meeting the traffic congestion problem are based mainly on decentralization, which basis is also urged by Mr. Bartholomew.** It is often difficult, however, to apply this principle without running counter to the natural tendency of the various kinds of business to herd together. To throttle this trend by drastic means, would be a long forward step toward traffic relief, but, on the other hand, the question might be raised as to how far such restriction could go without seriously hampering the quick and convenient prosecution of business.

Mr. Crosby†† refers to the arcaded "Rows" of Chester, England. Aside from their quaintness, these arcades are of special interest in this discussion, not because of the traffic relief they afford, but rather because of their popularity for business, notwithstanding that they are in second story (Fig. 17), as compared with that of frontage on the exterior sidewalk at grade.

Mr. Davison regards the "saturation point" in the use of the pleasure car as impending and urges the controlling of parking and the enforced use of by-passes, while Mr. Seabury stresses‡‡ the regulation of parking as affording a substantial relief. The writer does not concur in the view that the use of motor vehicles has reached its limit except where congestion has become so pronounced that driving is unpleasant, but believes that the relief of the congested districts would be at once followed by an increased use of motor vehicles in such areas. As to the relation of parking to the traffic problem in general,

* Discussion of the paper by Arthur S. Tuttle, M. Am. Soc. C. E., continued from November, 1924, *Proceedings*.

† Author's closure.

‡ Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

§ Received by the Secretary, October 24, 1924.

|| *Proceedings*, Am. Soc. C. E., May, 1924, p. 706.

¶ *Loc. cit.*, p. 713.

** *Loc. cit.*, p. 721.

†† *Loc. cit.*, p. 723.

‡‡ *Loc. cit.*, p. 725.

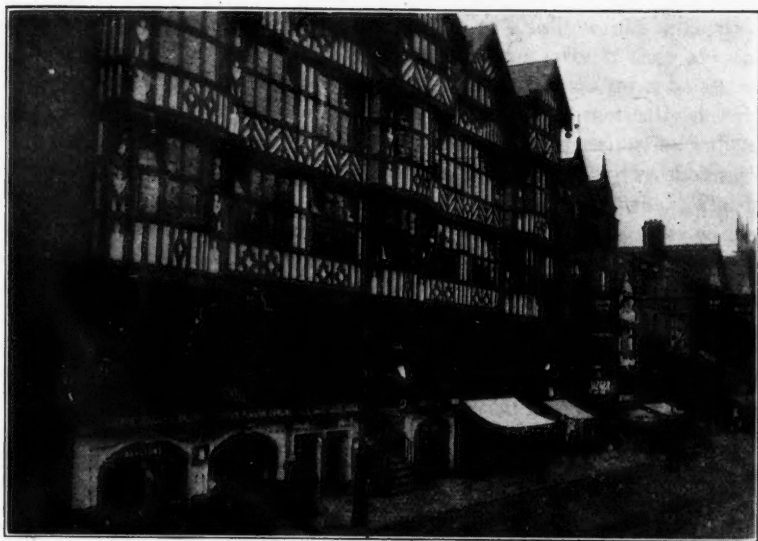


FIG. 17.—"THE ROWS", DUKE OF WESTMINSTER'S BUILDING, BRIDGE STREET, CHESTER, ENGLAND. CONTINUOUS ARCADE THROUGH SECOND FLOOR.

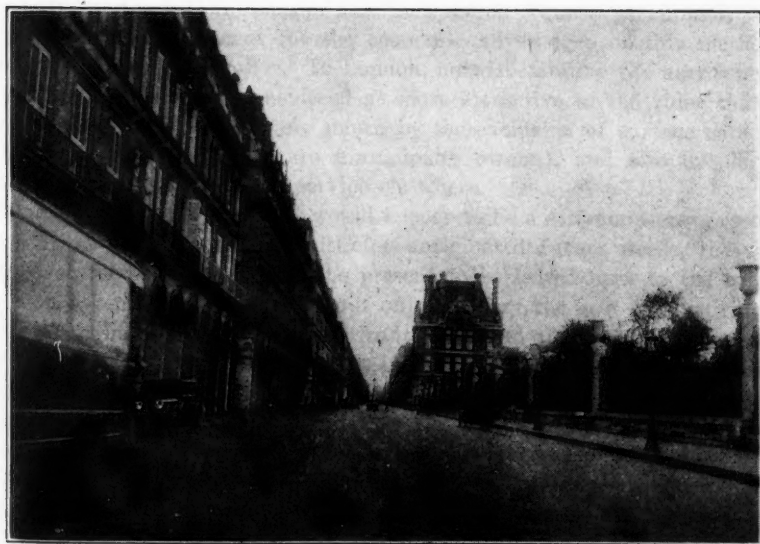


FIG. 18.—PARIS, FRANCE: RUE DE RIVOLI, OPPOSITE THE TUILERIES, LOOKING EAST.

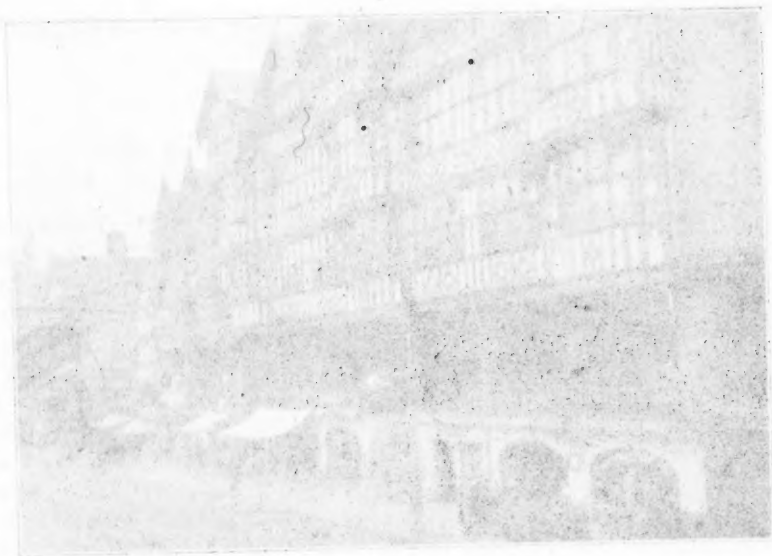


FIG. 17.—The House of Representatives, Washington, D. C., showing the main entrance and the large dome.



FIG. 18.—The House of Representatives, Washington, D. C., showing the main entrance and the large dome.

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there is much confusion of thought. The long continued occupancy of needed roadway space by a parked car cannot be condoned, but, on the other hand, the right to stop a car long enough for passengers to alight would seem to represent the minimum demand. Such stoppages, particularly if they are coupled with those incident to bus service (which the writer believes must be recognized as a coming serious factor in the use of public streets), practically destroy the value of the traffic lane nearest the curb except for by-passing, and then only where the roadway is insufficient for the accommodation of two moving lanes of vehicles of maximum width in addition to those at the curbs. The immediate prohibition of parking would seem to be a step so drastic as to demand caution; parking is essential if the roadways are to continue to make places of business accessible and cities attractive and thus more prosperous.

The platoon system proposed by Mr. Goodrich* is an ingenious device to obtain the advantages of a one-way street and yet permit travel in both directions. If car movement could be more closely regulated, this method could probably be adopted with advantage, but any irregularity would cause serious interference with cross traffic and possibly prevent its being successfully maintained.

Mr. Miller† contests the claim that in crowded districts the bus and taxicab have less objectionable effects on traffic than the surface car. The stopping of a trolley forces all other vehicles moving in the same direction to stop, whereas with bus service this is not the case, and, besides, the traffic lanes used by trolley cars are shunned by other classes of traffic owing to the continuous interruption of movement. In further support of the writer's views, attention might be called to the recent proposals made by the two largest traction companies in New York for the introduction of bus lines, which requests presumably would not have been made except for the economic advantages of this method of transportation over the trolley. In London, notwithstanding the narrowness of the roadways, busses are considered as more attractive to the rider than the tram. This view is conclusively shown by the exclusion of surface cars from the old city (although they are municipally owned), and also by the fast growing popularity of the bus service elsewhere.

Mr. Grunsky‡ suggests what would appear to be a common-sense procedure for securing adequate street width for anticipated future needs, the excess width over the requirements of the present to be leased back to the original owner on a rental basis sufficient not only to carry the cost of acquiring the excess width, but also partly to amortize it at the end of the rental period. The City of New York within recent years has secured legislation permitting a somewhat similar procedure. The excess width is first designated on the city plan as courtyard space, whereupon title may be taken by the city subject to the condition that the original owner may retain its use for a prescribed period common to all the parcels thus designated along the line of a street. It has been expected that property owners would be compensated for damage on the basis of the present worth of the award, this, in turn, being based on the probable value of the land and buildings at the time the city takes possession.

* *Proceedings, Am. Soc. C. E.*, August, 1924, p. 898.

† *Loc. cit.*, p. 899.

‡ *Loc. cit.*, November, 1924, p. 1513.

Thus, it was hoped that building damages in particular could be kept within reasonable limits.

The advisability, as well as the legality, of this procedure has been questioned, and its operation has been limited to a few test cases through which it is hoped to obtain conclusive evidence as to how the damages would be appraised by the Courts. In the first of these cases, buildings on the property were actually removed by the owner before the condemnation proceeding was completed in order to give way to an improvement of a more substantial character, and although payment for building damage was avoided, the question at issue was not settled. It is claimed by opponents of the plan that the procedure will cloud the title and thus make the land and buildings less readily salable. Even should the method eventually come into general use, it is evident that it will not solve the needs of the hour where congestion requires immediate relief.

Since the paper was written, the writer has made further investigations into the question of traffic congestion and remedies, including traffic counts in New York, and an examination of the Paris arcades and of traffic conditions in Paris and London. The seriousness of the situation in New York is perhaps best evidenced by the results of timing vehicular movement within an area extending approximately $\frac{1}{2}$ mile in all directions from the intersection of Fifth Avenue and 42d Street during the period of peak-load traffic. These tests show a speed for north and south movement, ranging from a maximum of 4.5 miles per hour on Fifth Avenue to a maximum of 12 miles per hour on First Avenue, averaging about 7.1 miles per hour; and for east and west movement, ranging from 4.75 miles per hour on 42d Street to 6.5 miles per hour on 44th Street, averaging about 5.7 miles per hour. In the writer's judgment, with proper car spacing regulated according to speed and under most favorable conditions, it is not practicable to move more than from 1 800 to 2 000 vehicles per lane per hour. Traffic on the Manhattan Bridge for 5-min. periods on successive occasions has been observed to exceed the rate of 1 400 vehicles per lane per hour without congestion, and if the terminals were adequate the maximum rate suggested could doubtless be reached. On the other hand, cross-traffic interference is responsible for cutting down the capacity in Fifth Avenue to an average of a little over 800 vehicles per hour per lane, and to an observed maximum for 5-min. periods of from 1 000 to 1 100 vehicles per hour per lane.

An extreme case of congestion and excess movement is presented at the viaduct over 42d Street on the line of Park Avenue in New York. Park Avenue has a roadway width of 72 ft.; practically all its traffic passes over the viaduct, which ranges in width from 36 to 40 ft. with two rectangular turns at the corners of the Grand Central Station. Traffic on the viaduct at the periods of maximum intensity moves in four lanes except in rounding the corners referred to where it is obliged to divide into two lanes. At the turn, the rate for 5-min. periods has exceeded 3 000 vehicles per hour for each lane.

Within the last few months the signal system has been extended to include a large number of the north and south arterial streets, parking has been curtailed in the congested sections, and most of the east and west streets south of Central Park have been made one-way streets. It would seem that little more can be accomplished without going beyond the limits of police measures.

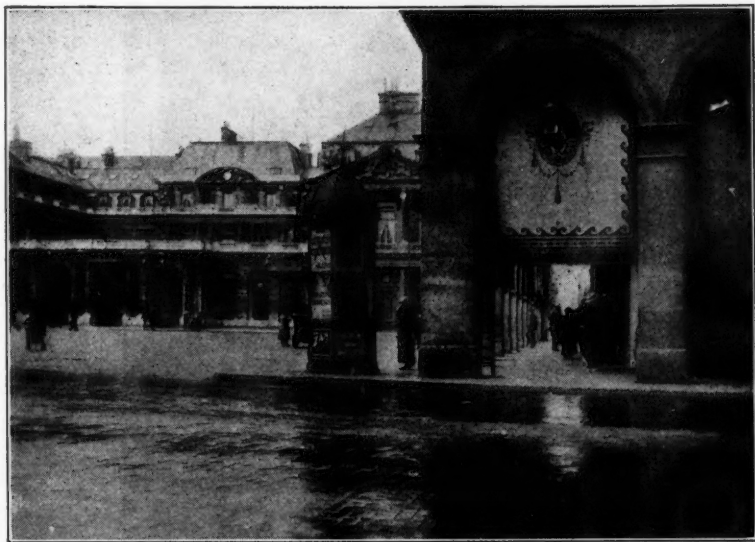


FIG. 19.—ARCADE THROUGH LOUVRE DEPARTMENT STORE, RUE DE RIVOLI, AT PALAIS ROYALE, PARIS, FRANCE. PROVISION HAS BEEN MADE FOR SIDEWALK EXTERIOR TO ARCADE.



FIG. 20.—PARIS, FRANCE: RUE DES PYRAMIDES, LOOKING THROUGH ARCADE FROM PLACE DE RIVOLI.



FIG. 20.—VIEW OF THE STREET FROM THE ARCHWAY, LOOKING EAST. THE ARCHWAY IS THE OLD MARKET PLACE, AND THE BUILDING ON THE RIGHT IS THE OLD MARKET PLACE.



FIG. 21.—VIEW OF THE STREET FROM THE ARCHWAY, LOOKING WEST. THE ARCHWAY IS THE OLD MARKET PLACE, AND THE BUILDING ON THE RIGHT IS THE OLD MARKET PLACE.

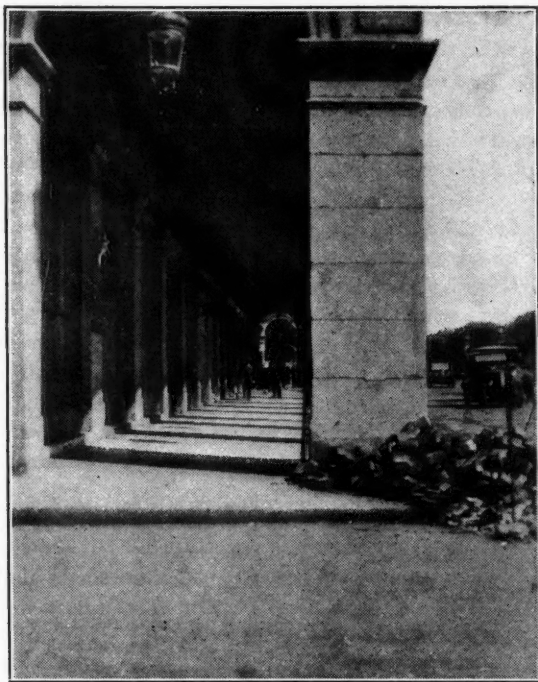


FIG. 21.—PARIS, FRANCE: LOOKING THROUGH THE RUE DE RIVOLI ARCADE.

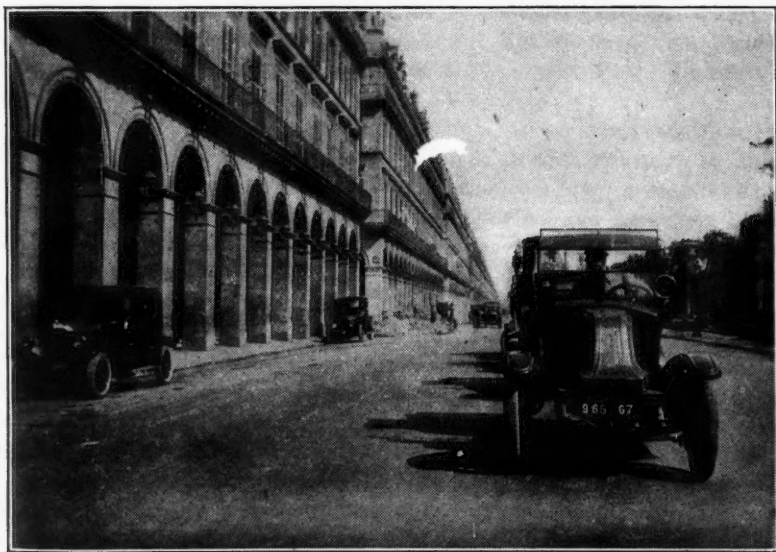


FIG. 22.—PARIS, FRANCE: RUE DE RIVOLI, LOOKING EASTWARDLY FROM RUE DE MONDOVI AT WEST END OF ARCADED SECTION.



THE GREAT BRITISH EMERALD JEWELRY AND DIAMOND EXHIBITION 1883



THE GREAT BRITISH EMERALD JEWELRY AND DIAMOND EXHIBITION 1883

In contrast to the annual rate of increase in traffic in New York, heretofore conservatively estimated by the writer as 15%, statistics recently furnished by the Traffic Department of the Metropolitan Police of London show a much lower rate, reaching only about 10% at Hyde Park corners (the Hyde Park end of Piccadilly, the point of maximum congestion). On the other hand, in old London, there is little yearly change, a condition due in part to the lack of street capacity and in part to the absence of the skyscraper. There traffic congestion is caused largely by the busses, the percentage of private cars being small in comparison with the United States and averaging for England only about 1 for every 40 persons. Traffic regulation has not begun to reach the stage obtaining in New York and is generally limited to the policing of all important intersections. The traffic officers are not assigned to a permanent post, nor are they retained continuously in this character of service as is the case here; although they are very efficient, it is manifest that under this system they cannot become as expert as the New York police. One-way traffic has long been in force during certain hours at congested points, and it is now being extended. The speed is limited to 20 miles per hour, except in congested areas, where 10 miles per hour is the limit. At present, the situation is considered sufficiently serious to warrant an investigation as to methods of bringing relief. Various plans are being offered for supplementing the few wide streets of London and attention is being directed to the comprehensive investigation made by the Royal Commission on London Traffic in 1905, which proposed a large number of projects designed to meet the needs of the congested parts of the city. The work done at that time evidently failed to receive support on the ground that it was too visionary.

In Paris, the narrower streets of earlier days have been relieved by a comprehensive system of boulevards. With a few exceptions, the traffic, which consists largely of taxicabs, flows freely through the city with little police supervision and apparently in the absence of a speed limit. In some of the narrower streets, one-way regulations are in force.

Paris probably affords the best example of securing additional street capacity through the introduction of sidewalk arcades, reference to which has already been made by the writer. Rue de Rivoli forms the eastwardly extension of Champs Elysées, one of the most attractive and capacious boulevards of the world; and, although it is flanked on each side with boulevards, it has a strategic position such as to make it one of the most important streets of the city. The exclusive character of the shops that line this street in the arcaded section, comprising a length of about $\frac{3}{4}$ mile, should remove the fears frequently expressed as to the depreciation of values due to the arcades, particularly when contrasted with those in the unarcaded section. Similar remarks as to values apply to Rue Castiglione and Rue des Pyramides, where the sidewalks on both sides are placed in arcades. (See Figs. 18, 19, 20, 21, and 22.)

As the skyscraper areas grow, it is quite clear that the acquisition of property for new streets becomes increasingly impractical and also that each new building increases the traffic load. The writer's later investigations lead him more strongly than ever to the conclusion that for such areas the ultimate remedy will be found in arcading the sidewalks and in adding the present sidewalk space to the roadways.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

ALEXANDER MOSBY CLAYTON BYERS, M. Am. Soc. C. E.*

DIED APRIL 9, 1924.

Alexander Mosby Clayton Byers was born on the family plantation called "Glenwood", near Batesville, Ark., on December 11, 1857, the son of Judge William Byers, a distinguished lawyer, and Emily Burton Byers. Genealogies of both sides of his family reveal the names of hero ancestors who served in all the American wars, including the Revolution.

Mr. Byers' education was acquired at private schools in Batesville, at Arkansas College, and at the University of the South, at Sewanee, Tenn. Among his instructors at the latter institution were the late Gen. William C. Gorgas, Gen. E. Kirby Smith, and Bishop Quintard.

Soon after the completion of his education Mr. Byers started work on that particular phase of engineering to which he gave most of his life—railway location and maintenance. His first work was with the old Kansas City, Springfield and Memphis Railroad Company. Immediately thereafter, during parts of 1879 and 1880, he was engaged in land surveying in Texas.

From 1881 to 1887, he was employed by the Mexican Central Railroad Company, successively, on location, construction, and maintenance. During the earlier part of this period, on three separate occasions, while he was on the preliminary surveys, Mr. Byers covered afoot the long distance from the City of Mexico to the United States Boundary.

For the next two years he was Assistant Engineer on the Panama Railroad, during the French régime, under the late Ferdinand de Lesseps. The failure of the canal project in 1889 seriously affected the railway, and Mr. Byers returned to Mexico, where he was immediately entrusted with the location and construction, for the Mexican Southern Railroad Company, of a line over exceedingly difficult terrain—the descent from the table-lands to the tropics. On the completion of this project, he rejoined the staff of the Mexican Central Railroad Company, and served for ten years as Engineer of the Maintenance Department of that line.

From 1902 to 1905, Mr. Byers was engaged in engineering and contracting work, mainly for the Dos Estrellas Mining Railroad Company in the State of Mexico, Mexico, and for the Vera Cruz and Pacific Railroad Company in the State of Vera Cruz, Mexico. Following these engagements, he was Chief Engineer of La Dicha and Pacific Railroad Company. Later, he became Superintendent of Tracks, Bridges, Buildings, and Water Service for the Tehuantepec National Railroad of Mexico.

Due to the troubled state of affairs in Mexico, Mr. Byers returned to the United States in 1914, and practiced his profession in Nevada. In 1915, he

* Memoir prepared by Walter F. Winton, Affiliate, Am. Soc. C. E.

again answered the call of the tropics and went to Guatemala, as Engineer on construction work and, later, during 1916 and 1917, to Cuba, returning to the United States on its entry into the World War. During the war, Mr. Byers made valuable confidential reports to the War Department.

For two years preceding his death, ill health compelled him to forego the active life to which he had been accustomed, and he therefore devoted his time to consulting work in San Francisco, Calif.

He was married on May 11, 1891, to Emma O. Stinson, who survives him.

The correspondence file left by Mr. Byers reveals the keen regret of his various employers when he left their service—a most striking testimonial of his technical ability.

He loved adventure and had traveled widely. He was a great hunter, and his reminiscences included tales of buffalo shooting on the plains of Texas. An intimate friend pays the following tribute: "Byers was a most companionable man. He loved a joke, saw the humor of all situations, was untiring physically and of a resilient spirit. His generosity was boundless. He tied his friends to him with strong bonds".

Throughout his life, Mr. Byers was a member of the Protestant Episcopal Church.

In the death of this kindly, talented, Christian gentleman, the Engineering Profession has sustained a great loss.

Mr. Byers was elected a Member of the American Society of Civil Engineers on November 6, 1907.

FRANK BARR KNIGHT, M. Am. Soc. C. E.*

DIED OCTOBER 12, 1924.

Frank Barr Knight was born in Worcester, Mass., on February 13, 1872, and was graduated from Worcester Polytechnic Institute in 1892, in the Department of Civil Engineering. A few weeks prior to his graduation he was engaged to enter the employ of the Lidgerwood Manufacturing Company which Company was his sole employer during his professional career, from 1892 until his death. He first entered the Cableway Department as a Draftsman and, later, was trained as a Cableway Erector. Mr. Knight's work brought him in contact with every phase of cableway design, construction, and operation. He made several improvements, notably in excavating buckets and aerial dumping appliances.

From 1894 to 1896, he was in charge of field construction on the Chicago Drainage Canal where many Lidgerwood cableways were used. In 1908, Mr. Knight was made Manager of the Chicago Branch of the Lidgerwood Manufacturing Company, in which position his talents for business management and engineering design were conspicuous factors in the success of his undertakings. He was the author of several articles on the subject of cableways, notably one entitled "Mining Lime Rock."†

* Memoir prepared by Spencer Miller, M. Am. Soc. C. E.

† *Mines and Minerals*, August, 1899.

Mr. Knight was President of the National Drainage Congress which met in Cairo, Ill., in January, 1916, having specialized in the study of drainage and reclamation projects, as well as equipment for construction and excavation.

He was a Thirty-Second Degree Mason and member of Kismet Temple, Brooklyn, N. Y. He was a member of the Union League Club and the Engineers Club of Chicago, the Western Society of Engineers, and the Machinery Club of New York, N. Y.

He died suddenly of heart failure on October 12, 1924, at his residence at Highland Park, Ill. For a great many months Mr. Knight had been in poor health, although he was able to attend to his business duties to the last.

He is survived by his wife, an only son, Frank Burrows Knight, and the host of friends whom he had attracted by his genial and generous nature.

Mr. Knight was elected an Associate Member of the American Society of Civil Engineers on September 4, 1901, and a Member on September 3, 1912.

CHARLES OSCAR MCCOMB, M. Am. Soc. C. E.*

DIED JULY 27, 1924.

Charles Oscar McComb, the son of Oscar and Isabel Tuck McComb, was born in New York, N. Y., on October 9, 1859. His education was acquired in that city and Brooklyn, N. Y., chiefly in private schools.

His first professional engagement was with the Pennsylvania Railroad Company, and from 1880 to 1884, he was employed in the Engineering Department of the Erie Railroad Company. In 1884, he engaged in private practice in Bloomfield, N. J., serving as Engineer for that village in 1885 and 1886. During part of 1886 and 1887, he was Assistant Engineer with the Carthage and Adirondack Railroad Company, to which, after a brief service with the Lehigh and Schuylkill Haven Railroad Company, he returned as Principal Assistant Engineer.

In 1889, Mr. McComb became Engineer and Superintendent of the Magnetic Iron Ore Company, which position he left to become associated with the firm of Hinds and Bond, Engineers and Contractors, in charge of the construction and operation of water-works at Chatham, Ont., Canada, until 1892. In that year, he resumed his private practice in Carthage, N. Y. He designed and built water-works and sewerage systems for a number of cities and villages in Northern New York, water-power developments on the Black, Oswegatchie, and Grass Rivers, and relocated part of the Gouverneur and Edwards Railroad. In 1894, he became City Engineer of Watertown, N. Y., and, in 1898, also Engineer of the Water Board of that city. From that time until 1904, considerable paving, sewer construction, and large bridge work was done under his direction.

Mr. McComb relinquished this position to become Division Engineer of the State of New York, with headquarters at Syracuse. During his occupancy of this office, he had charge of both canal and highway work. Foreseeing the

* Memoir prepared by J. H. Weldman, Assoc. M. Am. Soc. C. E.

great expansion in highway construction, he entered this field of contracting on his own account in 1907, and was immediately successful. This occupation was continued until his death, except for the interruption of the World War period, when road building was suspended. During this time, he was engaged as Chief Engineer for the contractor on the construction of the Housing Project at Watertown, N. Y., under the Housing Bureau of the United States Department of Labor.

Mr. McComb had an agreeable and pleasant personality which gained him many friends, his scrupulously honorable and conscientious character enabling him to retain them through life. He was a loyal member of the Protestant Episcopal Church and, in politics, of the Republican Party.

In 1890, Mr. McComb was married to Caroline E. Benton, of Adams, N. Y., who survives him.

Mr. McComb was elected a Member of the American Society of Civil Engineers on May 2, 1900.

LOCKE MCINDOE PERKINS, M. Am. Soc. C. E.*

DIED JULY 2, 1924

Locke McIndoe Perkins, the son of Marsh Olin and Clara McIndoe Perkins, was born at Windsor, Vt., on November 20, 1879. He entered Dartmouth College in 1897, from which he received the degree of Bachelor of Science in 1901, and the degree of Civil Engineer (Thayer School) in 1903.

Beginning in April, 1903, Mr. Perkins served the Northern Pacific Railway Company as Rodman and as Instrumentman on the construction of new terminals in Seattle, Wash., for two years. In April, 1905, he was promoted to the position of Resident Engineer on the construction of the Sykeston Branch Extension, and a few months later was again promoted and transferred to Duluth, Minn., as Assistant Engineer in Charge of Maintenance of the Lake Superior Division. In November, 1906, he returned to the Construction Department as Assistant Engineer on second-track line and grade revision between Garrison and Missoula, Mont., and the relocation of the White Pine Hill Line. In April, 1907, he was promoted to the position of Division Engineer, in charge of maintenance of the Northern Pacific, Lines East of Mandan, N. Dak. In February, 1910, he was made Engineer, Maintenance of Way, of all Lines East of Paradise, Mont., in general charge of maintenance and construction work, and in June, 1911, he was transferred to Tacoma, Wash., as Engineer, Maintenance of Way, of all Lines West of Paradise.

During this period, Mr. Perkins had general charge of the construction of new terminals and yards at Parkwater, Pasco, Auburn, Centralia, and Seattle (Hanford Street), Wash., as well as of a number of yard extensions, depots, etc., in addition to general supervision of all maintenance work.

In August, 1918, Mr. Perkins was made Corporate Engineer, in charge of the interests of the Northern Pacific Railway Company in matters of main-

* Memoir prepared by H. E. Stevens, M. Am. Soc. C. E.

tenance and construction during the period of Federal Control. In this capacity, he assisted in establishing a number of important fundamental principles used as a basis for the general settlements, between the Government and the carriers, of the claims arising from Federal Control and the Guaranty Period.

In March, 1920, he was made Assistant to the Chief Engineer, and in January, 1921, was promoted to the position of Executive Assistant to the President, continuing in this office until his death.

Mr. Perkins possessed to an unusual degree the keen, analytical mind necessary for the successful engineer. His fair and considerate dealing with his subordinates and with the many departments of the Railway endeared him to all his associates.

In private life, his kindly disposition and tactfulness made him a host of friends and a leader in movements for civic betterment. He was particularly interested in helping the working boy improve his position; he assisted in organizing and acted as one of the first leaders of the Northern Pacific Employed Boys Club.

On February 11, 1908, Mr. Perkins was married to Ruth Roberts, of Devils Lake, N. Dak., who, with three sons, Locke, Donald, and Marsh, and one daughter, Ruth, survives him.

He was a member of the St. Paul Athletic Club, the Town and Country Club, the Benevolent Protective Order of Elks and of the Masonic Fraternity. Mr. Perkins was elected a Member of the American Society of Civil Engineers on September 9, 1919, and was prominent in the activities of the Northwestern Section.

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of the

American Society

of

Civil Engineers

(INSTITUTED 1852)

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ON CONCRETE AND REINFORCED CONCRETE ARCHES: C. T. Morris, G. E. Beggs, J. R. Chamberlin, E. H. Harder, A. C. Janni, W. M. Wilson.

ON EFFECTS OF EARTHQUAKES ON ENGINEERING STRUCTURES: J. D. Galloway, Frederick H. Fowler, John Mills, C. H. Snyder, C. B. Wing. SUB-COMMITTEE: Isami Hiroi, Mikishi Abe, Hyotaro Inagaki, Masayoshi Kabashima, Tashiro Shiraishi.

ON ELECTRIFICATION OF STEAM RAILWAYS: Charles F. Loweth, Bion J. Arnold, George Gibbs, George W. Kittredge, E. J. Pearson, Samuel Rea, Robert Ridgway.

ON STRESSES IN STRUCTURAL STEEL: F. O. Dufour, H. G. Balcom, Clement E. Chase, O. F. Dalstrom, J. H. Edwards, Robert Farnham, R. J. Fogg, F. M. Masters, L. D. Rights, F. E. Schmitt, W. J. Thomas, L. J. Towne.

ON IMPACT IN HIGHWAY BRIDGES: A. H. Fuller, A. R. Eltzen, E. F. Kelley, F. E. Turneure.

ON FLOOD-PROTECTION DATA: N. C. Grover, C. B. Burdick, W. P. Creager, H. P. Eddy, Gerard H. Matthes, Charles H. Paul, A. O. Ridgway.

ON IRRIGATION HYDRAULICS: D. C. Henny, W. F. Allison, B. A. Etcheverry, Samuel Fortier, R. L. Parshall, J. L. Savage, F. C. Scobey, Stuart Sims, J. C. Stevens, Franklin Thomas.

ON HYDRAULICS PHENOMENA: S. M. Woodward, M. L. Enger, R. E. Horton, A. T. Safford, E. W. Schoder.

ON STEEL COLUMN RESEARCH: F. E. Turneure, C. G. E. Larsson, B. R. Leffler, G. L. Taylor, S. H. Widdicombe.

ALFRED NOBLE MEMORIAL COMMITTEE: Samuel Rea, Bion J. Arnold, Onward Bates, J. Vipond Davies, George Gibbs, William W. Harts, S. H. Hedgus, J. W. Lieb, F. H. Newell, Robert Ridgway, J. Waldo Smith.

* Secretary Dunlap died July 29, 1924.

† Appointed Director April 7, 1924, to fill the vacancy caused by the resignation of R. N. Beglen.

‡ Director Holland died October 27, 1924.

AMERICAN SOCIETY OF CIVIL ENGINEERS

COMING MEETINGS

BOARD OF DIRECTION MEETINGS

January 19-20, 1925:

A Quarterly Meeting will be held at New York, N. Y.

MONTHLY MEETINGS

December 10, 1924:

8:00 P. M.—A regular business meeting of the Society will be held, and a paper by William M. Black, M. Am. Soc. C. E., entitled "Waterway and Railway Equivalents", will be presented for discussion. This paper was published in *Proceedings* for August, 1924.

ANNUAL MEETING, NEW YORK, N. Y.

January 21, 22, and 23, 1925:

Seventy-second Annual Meeting of the Society.

January 21, 1925:

9:00 A. M.—Social Hour.

10:00 A. M.—Annual Meeting and Presentation of Medals and Prizes for Papers.

2:30 P. M.—Presentation and Discussion of Committee Reports.

7:30 P. M.—President's Reception and Dinner Dance.

January 22, 1925:

9:00 A. M.—Social Hour.

10:00 A. M. to 4:30 P. M.—Meetings of Technical Divisions.

8:00 P. M.—Address and Smoker.

January 23, 1925:

All-Day Excursion.

The Reading Room of the Society is open from 9 A. M. to 6 P. M. every day, except Sundays, New Year's Day, Washington's Birthday, Memorial Day, Fourth of July, Labor Day, Thanksgiving Day, and Christmas Day; during July and August it is closed at 5 P. M.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it as a place for meeting others. There is a file of 260 current periodicals, the latest technical books, and the room is well supplied with writing tables.

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